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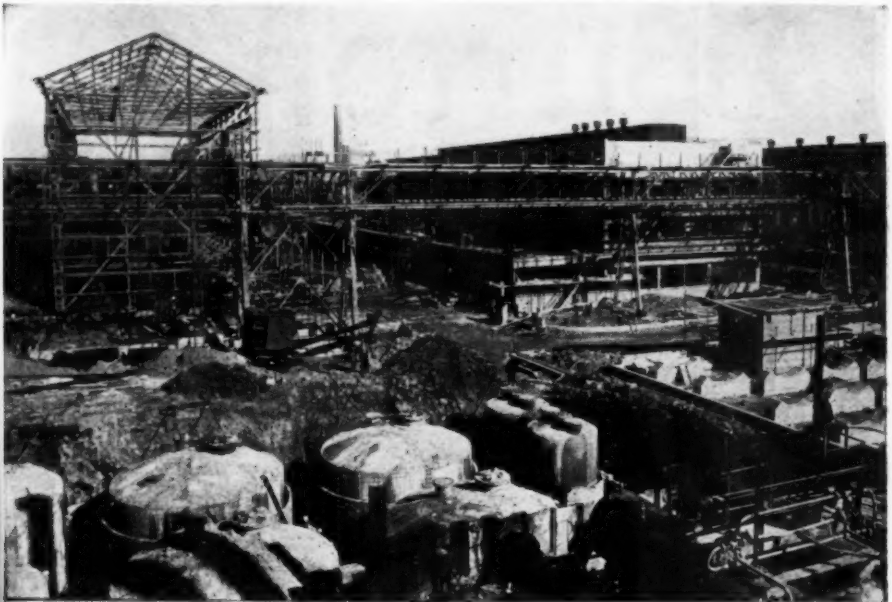

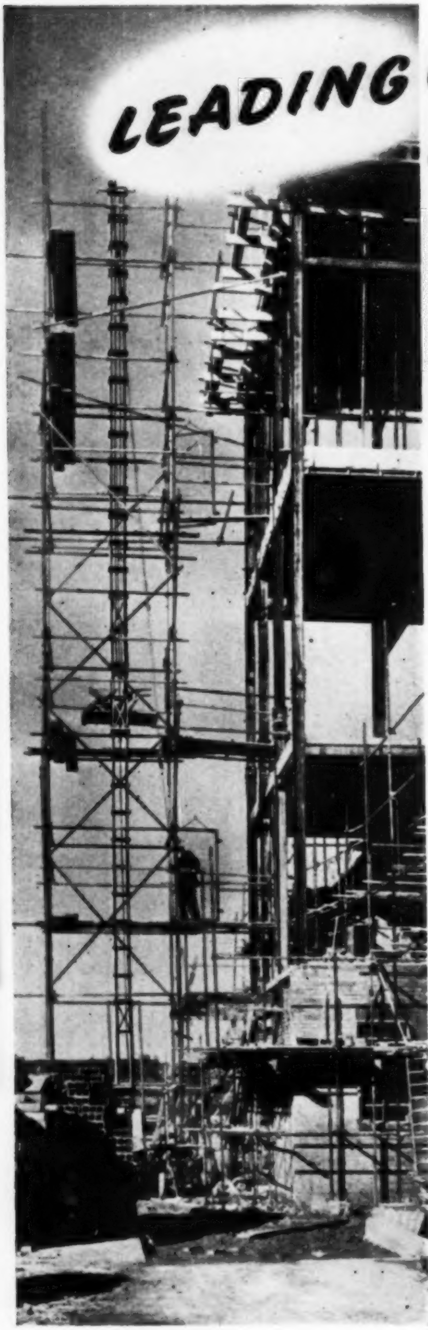

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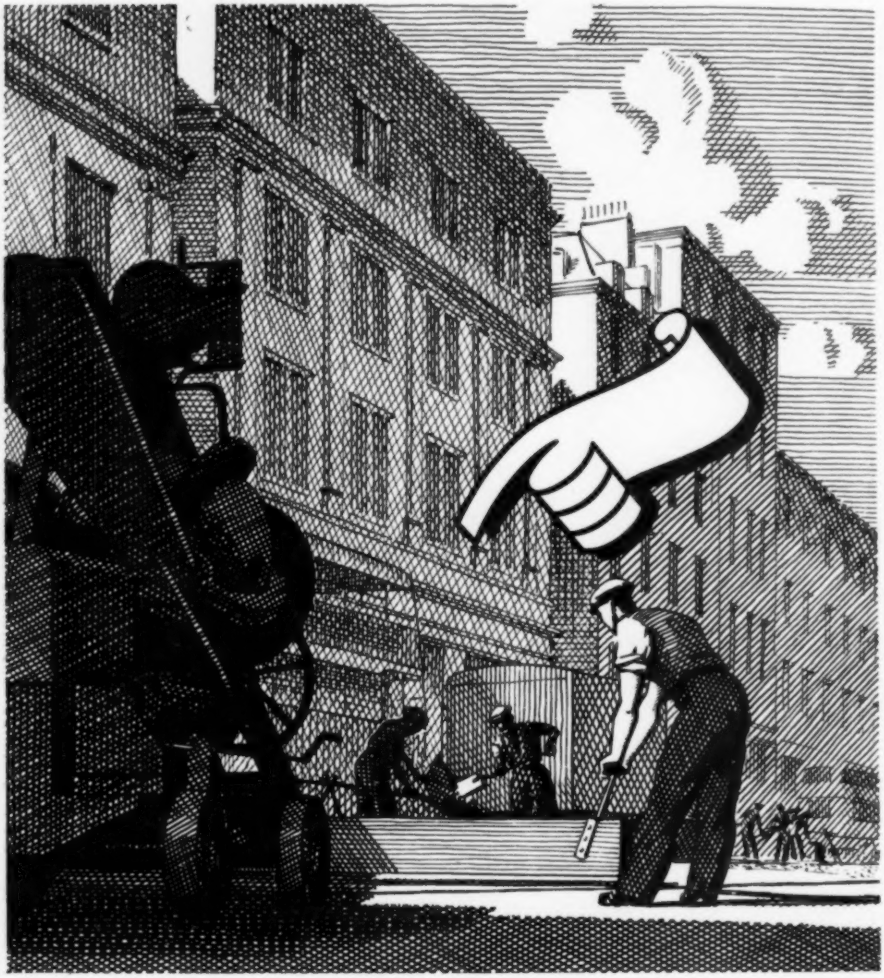

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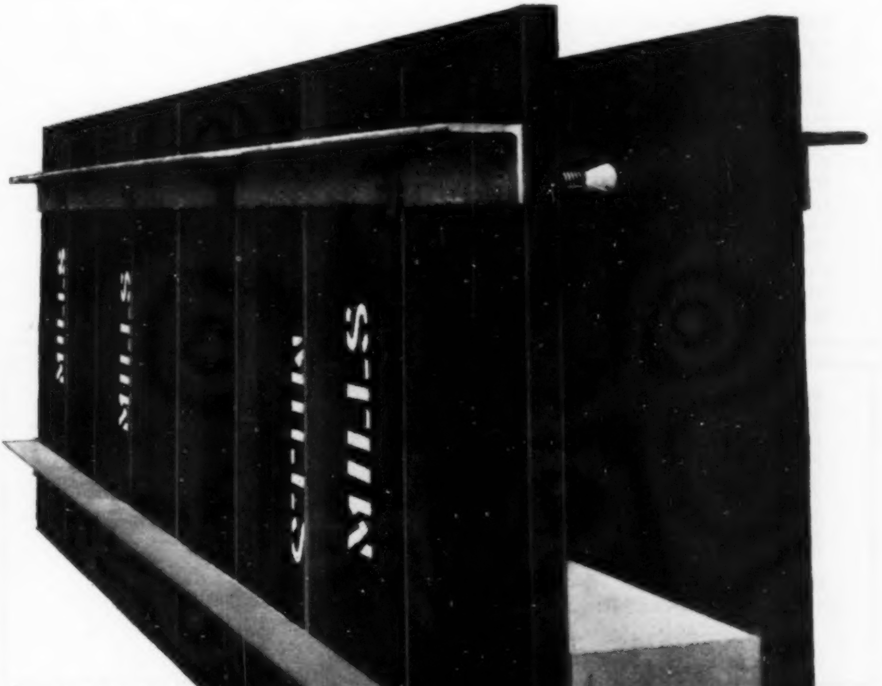

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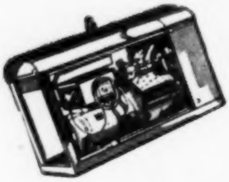

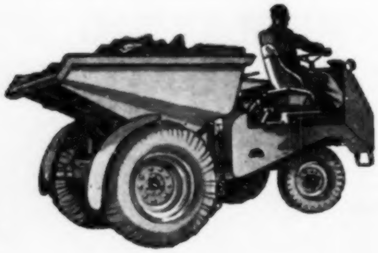

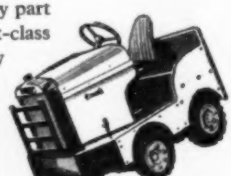

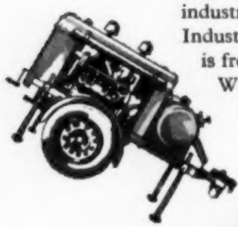

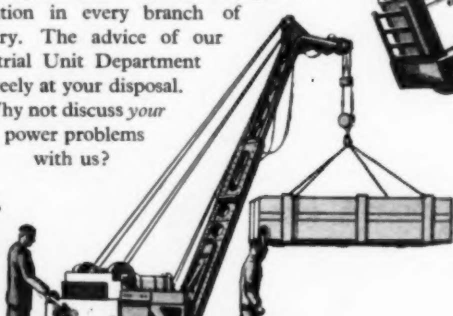

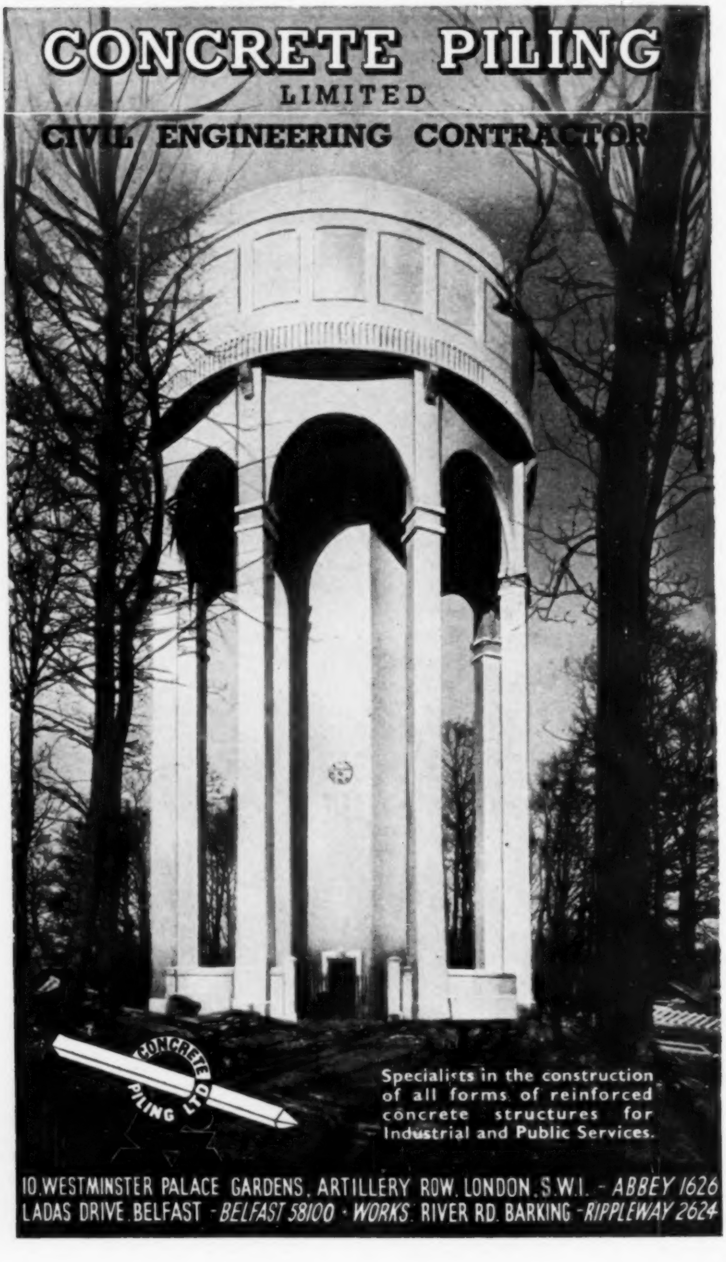
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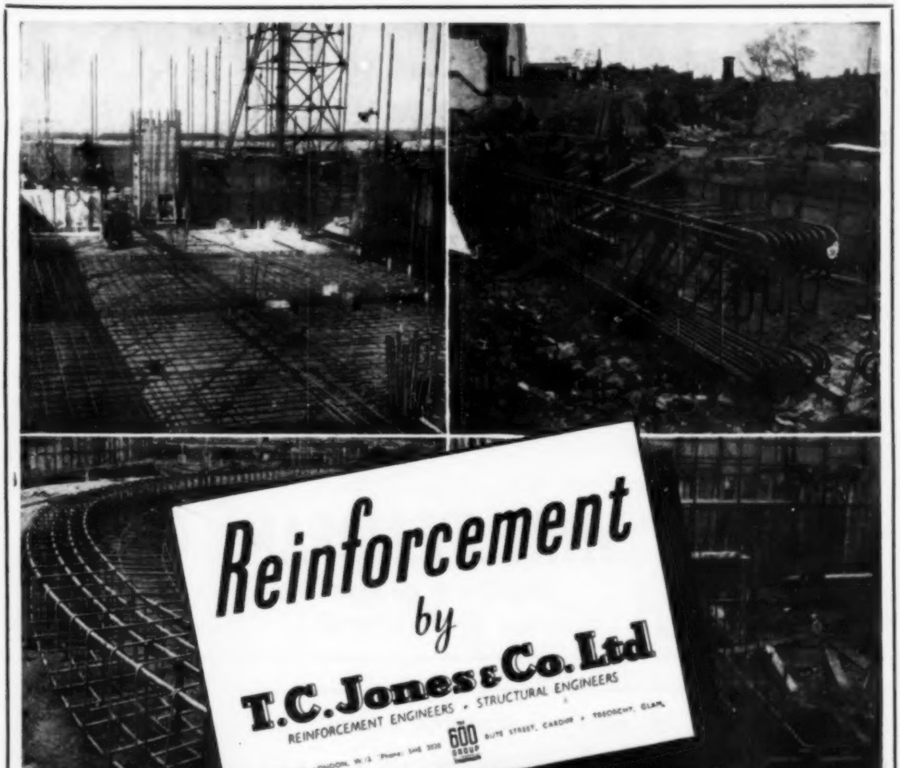
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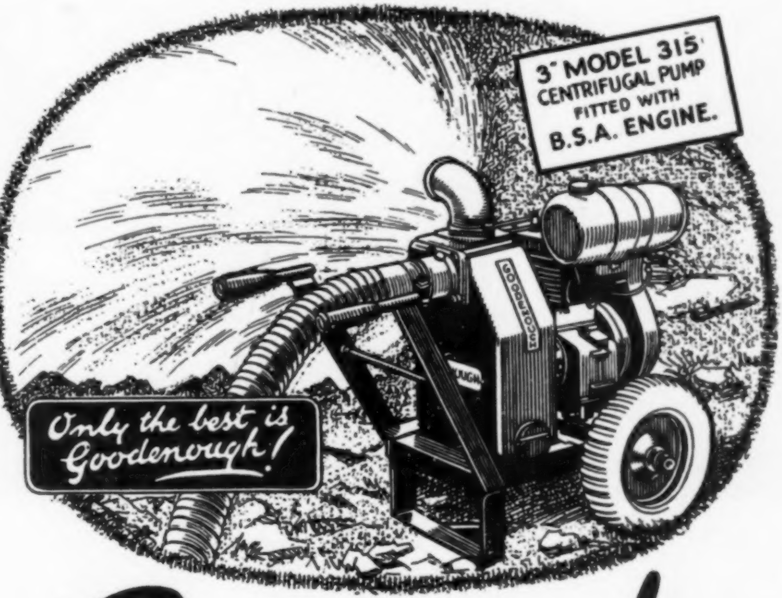
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Volume XLVII. No. 7. LONDON, JULY, 1952

EDITORIAL NOTES

Aeronautics and Civil Engineering.

AT first sight there does not seem to be a very close connection between concrete construction and aircraft design, and probably not many civil engineers engaged on research, design, or civil engineering construction realise that without their work high-speed flight would be impossible. This was one of the points made in the last Wilbur Wright Memorial Lecture * to the Royal Aeronautical Society. The phenomenal performance of present-day aircraft, the lecturer said, had not been exclusively due to those who actually designed and built the aeroplanes, for the best designers of to-day could do very little better than the Wright brothers if they were limited to the materials, constructional methods, and equipment of fifty years ago. Explaining the importance of constructional engineering to air transport, the lecturer referred to a calculation made by Hermann Glauert, one of the most brilliant British aeronautical scientists of the 1920's, "who established the thesis that the stalling speed of an aeroplane could not be less than about one-quarter or one-fifth of its maximum flying speed, and no means had then been devised for landing aircraft at lower speeds. This meant that landing speeds in excess of a hundred miles an hour were adopted from necessity. Long concrete runways were therefore no less essential to the supersonic aircraft of to-day than the jet engines which propelled them." The resources of constructional engineering are also at the service of atomic scientists in the design of concrete shields to confine the extraneous nuclear radiation which cannot at present be controlled and directed to the production of useful work.

It would be unrealistic to suppose that the place of concrete in the development of atomic energy is any more than a minor and subsidiary adjunct of the atomic piles and reactors, just as in conventional power stations the steam turbines and the generators are the vital elements rather than the chimneys and cooling towers. However, this subordination of the constructional element to the mechanical, electrical, and scientific must not lead civil engineers to restrict their interest to structures in which the constructional features predominate. It is as necessary that research should be directed to reducing the thickness of a runway, or of an atomic screen, as it is to increase what are now considered to be the

* "Prophecy and Achievement in Aeronautics." 40th Wilbur Wright Memorial Lecture, read before the Royal Aeronautical Society at the Institution of Civil Engineers by Sir Harry M. Garner, K.B.E., C.B., M.A., Chief Scientist, Ministry of Supply.

longest economic span of a girder or height of a column. For this reason, designers of runways and the ancillary equipment of mechanical and chemical engineering plant will be grateful for the recognition accorded in this lecture to their work on airfields.

The extent of a high-speed aeroplane's dependence on civil engineering was summarised by the lecturer when he referred to long and expensive runways as " stupendous millstones around the necks of military and civil operators ", which were proving an intolerable burden. Indeed so heavy was this burden that those concerned with flying were already considering the possibilities of a new type of flying machine capable of taking-off and landing vertically, a development which would depend on the production of an engine thrust greater than the weight of the aircraft. Another method of avoiding expensive runways is, of course, a greater dependence on flying boats, especially for civil transport. " The construction of larger and larger costly land aerodromes, using land that would otherwise be available for agriculture and industrial use, must surely be found to be too great a price to pay for the convenience of civil aviation." If, as the lecturer envisaged, the limit has been reached in conventional runway construction, new constructional problems will arise in connection with the operation of novel types of land plane and larger flying boats, and constructional engineers must be ready to meet and solve these problems as they arise.

Discussing the comparatively recent achievement of supersonic flight the lecturer said : " The excessive rise in ' drag ' as the speed of sound is approached, with the associated loss of stability and control, led to what was known as the ' brick-wall barrier '. Scientists thought that not only would flight at speeds beyond that of sound be difficult, but that flight in the region near that of sound would be highly unstable and dangerous. If sufficient thrust could be provided to enable the aircraft to pass through this dangerous region quickly there might not be time for the lack of stability and control to make itself felt. But there was doubt whether this passage of the barrier would ever be more than a dangerous experimental technique." That was the problem which confronted aeronautical scientists only a few years ago. Now the problem has been solved, and the lecturer continued : " Subsequent developments have shown that the difficulties of transonic and supersonic flight are not so serious as the scientists had thought." Here is a lesson to be learned and an example to be marked not only by those engaged in aeronautics but by scientists and engineers in all branches of engineering. A " brick-wall barrier ", which appeared to be an insurmountable obstacle, later lost its apparent invincibility. Constructional engineers have a long-standing and continuing reputation for their refusal to be daunted by difficulties, as the achievements of I. K. Brunel, Sir Benjamin Baker, Sir Donald Bailey, and many others bear witness, but it is well to be reminded by one of the foremost authorities in another branch of endeavour and achievement that there are few " brick-wall barriers " that cannot be demolished or surmounted by the scientist or the engineer if he bring to his task a sufficient measure of confidence, determination, and skill.

Rectangular Slabs Spanning in Two Directions.

Analysis by Pattern of Fractures.

By Professor H. CRAEMER (ALEXANDRIA).

THE principles of the analysis of a reinforced concrete slab supported along two or more sides were dealt with by the writer in this journal for August, 1950. In the following, these principles, which are based on a consideration of the pattern of the fractures at failure of the slab, are applied to rectangular panels supported on four sides and carrying a uniformly-distributed load or a trapezoidal load, and a slab supported on three sides carrying a triangularly-distributed load.

Slab supported on Four Sides: Uniformly-distributed Load.

The pattern of the fractures of a rectangular panel has been frequently shown by tests to be as in *Fig.* 1, the fragments being ABFE, CDEF, AED,

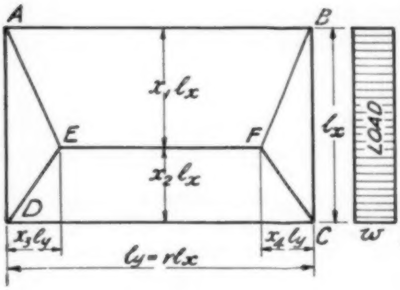


Fig. 1.

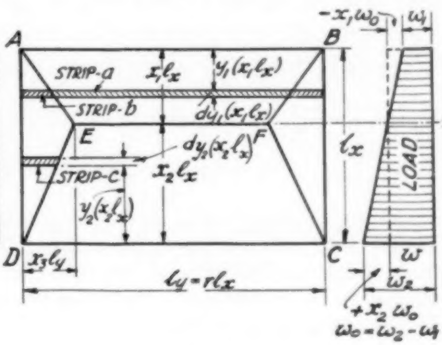


Fig. 2.

and BCF, formed by fractures along the lines AED, BFC, and EF. Sometimes the fractures fork near the corners, but the effect on the load-carrying capacity of the slab is negligible. In *Fig.* 1, l_x and l_y are the lengths of the sides of the panel of a slab of constant thickness, and $\frac{l_y}{l_x} = r$. The reinforcement in the bottom of the slab is assumed to produce a moment of resistance m_x at failure for the bars parallel to l_x, and m_y for the bars parallel to l_y; m_x and m_y are the moments for unit length. If there is reinforcement in the top of the slab at the supports, the lines of the supports are also lines of fracture and the corresponding moments of resistance are m_x', m_x'', m_y', and m_y'' on unit length of AB, CD, AD, and BC respectively.

Generally, the position of the fracture EF is not known, that is the dimensions $x_1 l_x$, $x_2 l_x$, $x_3 l_y$, and $x_4 l_y$ are unknown. Consider fragment ABFE. Since the bottom reinforcement parallel to l_x is generally constant for the distance l_y, it exerts a moment $m_x l_y$ upon the fragment, and the reinforcement along the top

of the edge AB exerts a moment $m'_x l_y$. The total moment of the internal forces about AB is therefore $(m_x + m'_x)l_y$. If the intensity of the uniformly-distributed load on the slab is w, the total moment about AB of the load on the fragment is

$$[0{\cdot}5x_1^2l_x^2l_y - 0{\cdot}33x_1^2l_x^2(x_3 + x_4)l_y]w,$$

which must be equal to the total moment of the internal forces, that is

$$m_x + m'_x = \frac{wx_1^2l_x^2}{6}[3 - 2(x_3 + x_4)] = C_1wl_x^2 \quad . \quad . \quad . \quad (1)$$

Similarly, for fragment CDEF,

$$m_x + m''_x = \frac{wx_2^2l_x^2}{6}[3 - 2(x_3 + x_4)] = C_2wl_x^2 \quad . \quad . \quad . \quad (2)$$

For the triangular fragment ADE, equating the total moment of the internal forces $(m_y + m'_y)l_x$ to the moment of the load about AD, $\frac{wl_xx_3^2l_y^2}{6}$, gives

$$m_y + m'_y = \frac{wx_3^2l_y^2}{6} = K_1wl_x^2 \quad . \quad . \quad . \quad . \quad (3)$$

Similarly, for fragment BCF, $m_y + m''_y = \frac{wx_4^2l_y^2}{6} = K_2wl_x^2 \quad . \quad . \quad . \quad (4)$

The coefficients in formulæ (1) to (4) are

$$C_1 = \frac{x_1^2}{6}[3 - 2(x_3 + x_4)]\,; \quad C_2 = \frac{x_2^2}{6}[3 - 2(x_3 + x_4)]\,; \quad K_1 = \frac{r^2x_3^2}{6}\,; \quad K_2 = \frac{r^2x_4^2}{6}.$$

A chart giving values of C_1 and C_2 is given in *Fig.* 3. If a pattern of cracks is assumed, that is if values of the ratios x_1 to x_4 are assumed, the corresponding values of the coefficients and the value of wl_x^2 enable the sums of the bending moments to be calculated. The moments m'_x and m''_x at the supports occur only with the midspan moments m_x, and m'_y and m''_y only with m_y. The total resistance, therefore, remains unchanged if the bottom reinforcement is increased and the top reinforcement decreased, or inversely.

In the numerical examples which follow the method of designing slabs for various conditions is described, and in the first example a comparison is made with the results obtained by applying the recommendations of the British Standard Code of Practice No. 114.

Example No. 1.—The dimensions of a panel continuous over the supports on four sides are: l_x, 16·7 ft. and l_y, 17·8 ft.; w = 350 lb. per square foot. (*a*) Assume $x_1 = x_2 = 0{\cdot}5$, and $x_3 = x_4 = 0{\cdot}4$; therefore $x_3 + x_4 = 0{\cdot}8$. Since $r = \frac{17{\cdot}8}{16{\cdot}7} = 1{\cdot}06$, from *Fig.* 3 $C_1 = C_2 = 0{\cdot}0583$. Therefore

$m_x + m'_x = m_x + m''_x = 0{\cdot}0583wl_x^2 = 0{\cdot}0583 \times 350 \times 16{\cdot}7^2 = 5700$ ft.-lb. per foot. If the bending moment at the support is assumed to be 3200 ft.-lb., the bending moment at midspan is 5700 − 3200 = 2500 ft.-lb. in the direction of l_x. By substitution, $K_1 = K_2 = \frac{(1{\cdot}06 \times 0{\cdot}4^2)}{6} = 0{\cdot}03$, and, from (3) and (4), $m_y + m'_y = m_y + m''_y = 0{\cdot}03 \times 350 \times 16{\cdot}7^2 = 2920$ ft.-lb. per foot. If the

bending moment at the support is assumed to be 1750 ft.-lb., the bending moment at midspan is 2920 − 1750 = 1170 ft.-lb. in the direction of l_y. Thus all the bending moments required for the design are known. By symmetry, $x_1 = x_2 = 0{\cdot}5$ is an accurate assumption, but $x_3 (= x_4)$ might be more or less than the assumed value of 0·4. Any other value assumed, as in (*c*), will affect the calculated bending moments, but, if the resistances required by any reasonable assumption are provided, the design will be sound. This view also applies to the division of the total bending moment between the sections at the supports and midspan. All bending moments are on 1-ft. width of slab.

(*b*) In accordance with the recommendations of British Standard Code No. 114, the bending moments in the foregoing are as follows:

$m_x = 0{\cdot}028 \times 350 \times 16{\cdot}7^2 = 2740$ ft.-lb.; $m'_x = m''_x = 0{\cdot}037 \times 350 \times 16{\cdot}7^2 = 3600$ ft.-lb.

$m_y = 0{\cdot}025 \times 350 \times 16{\cdot}7^2 = 2440$ ft.-lb.; $m'_y = m''_y = 0{\cdot}033 \times 350 \times 16{\cdot}7^2 = 3230$ ft.-lb.

These bending moments exceed those calculated by the proposed method, but resistance to them need be provided only on a width of slab equal to three-quarters of the span, whereas in the design based on the pattern of fractures it is assumed that the resistance is provided for the entire length and breadth of the panel.

(*c*) Assume $x_3 = x_4 = 0{\cdot}2, x_3 + x_4 = 0{\cdot}4$; then $m_x + m_x = m_x + m''_x = 0{\cdot}092wl_x^2$ compared with $0{\cdot}0583wl_x^2$ in (*a*) and $(0{\cdot}028 + 0{\cdot}037) = 0{\cdot}065wl_x^2$ in (*b*). Also,

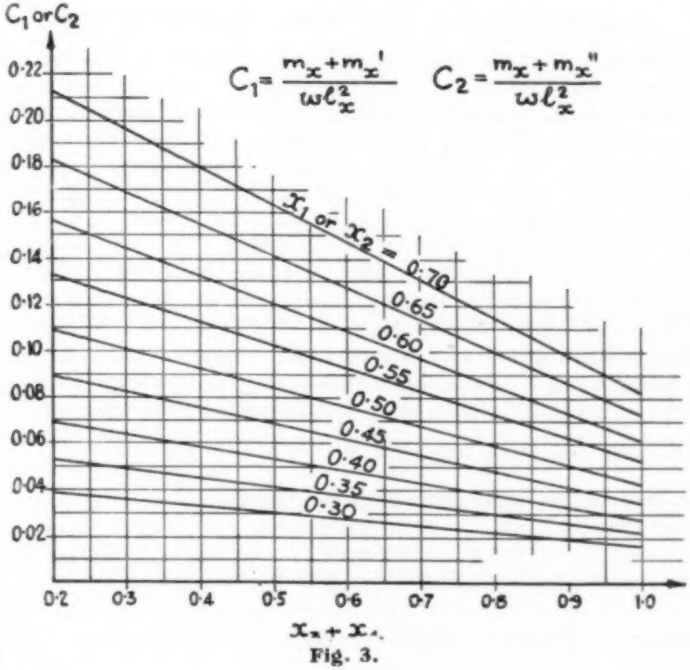


Fig. 3.

$m_y + m_y = m_y + m_y'' = 0{\cdot}007wl_x^2$ compared with $0{\cdot}030wl_x^2$ in (*a*) and $(0{\cdot}025 + 0{\cdot}033) = 0{\cdot}058wl_x^2$ in (*b*).

Example No. 2.—Assume that the slab in Example No. 1 is freely supported along one of the shorter sides. Assume that $x_1 = x_2 = 0{\cdot}5$, $x_3 = 0{\cdot}35$, and $x_4 = 0{\cdot}45$; $x_3 + x_4 = 0{\cdot}8$ and $C_1 = C_2 = 0{\cdot}0583$, and, as before, $m_x + m_x' = 5700$ ft.-lb. per foot, $m_x' = m_x'' =$, say, 3320 ft.-lb., and $m_x = 2380$ ft.-lb. Also, $K_1 = 0{\cdot}023$, and $m_y + m_y' = 0{\cdot}023 \times 350 \times 16{\cdot}7^2 = 2250$ ft.-lb.; $K_2 = 0{\cdot}038$, and $m_y + m_y'' = 0{\cdot}038 \times 350 \times 16{\cdot}7^2 = 3710$ ft.-lb. Since $m_y' = 0$, $m_y = 2250$ ft.-lb., and $m_y'' = 3710 - 2250 = 1460$ ft.-lb. per foot.

Example No. 3.—It is assumed that the reinforcement over the supports of the slab in Example No. 1 is such that $m_x' = 4410$ ft.-lb. and $m_x' = 1770$ ft.-lb. per foot. Therefore $\dfrac{m_x' - m_x''}{wl_x^2} = \dfrac{4410 - 1770}{350 \times 16{\cdot}7^2} = 0{\cdot}0272$. From (1) and (2), $\dfrac{m_x' - m_x''}{wl_x^2} = \dfrac{2l_xx_1 - l_x^2}{6l_x^2}[3 - 2(x_3 + x_4)]$. If it is assumed that $x_3 = x_4 = 0{\cdot}4$, from the two preceding equations $x_1 = 0{\cdot}558$ and $x_2 = 0{\cdot}442$, for which $C_1 = 0{\cdot}073$ and $C_2 = 0{\cdot}046$. Therefore $m_x + m_x' = 0{\cdot}073 \times 350 \times 16{\cdot}7^2 = 7100$ ft.-lb. and $m_x + m_x'' = 0{\cdot}046 \times 350 \times 16{\cdot}7^2 = 4450$ ft.-lb., from which $m_x = 7100 - 4410 = 2690$ ft.-lb., or $m_x = 4450 - 1770 = 2680$ ft.-lb. per foot.

Slab supported on Four Sides: Trapezoidal Load.

It is assumed that the intensity of load is constant in the direction l_y but varies from w_1 along AB to w_2 along CD (*Fig.* 2). The load can then be considered as a uniformly-distributed load w and a double triangularly-distributed load, the ordinate at CD being $w_2 - w$ (positive) and the ordinate at AB being $w - w_1$ (negative). The load w is the intensity of load at the fracture EF, and is therefore $0{\cdot}5[w_1 + w_2 + (x_1 - x_2)w_0]$ if $w_0 = w_2 - w_1$. The action of w is similar to that described in the preceding part of this article. The action of the triangular loadings is considered in the following. The two actions must be superposed to obtain the resultant effect. Within limits, the values of x_1 to x_4 can be chosen freely as before, but the same pattern must be assumed for the two actions since they occur simultaneously.

It is seen that the maximum intensity of the negative load is x_1w_0 and of the positive load x_2w_0 as in *Fig.* 2. The load on an elementary strip a of fragment ABFE is $-(1 - y_1)[1 - y_1(x_3 + x_4)]x_1w_0l_y.dy_1(x_1l_x)$, and the total moment about AB of the loads on all strips from $y_1 = 0$ to $y_1 = 1$ is $\dfrac{x_1^3w_0l_x^2l_y}{12}[2 - (x_3 + x_4)]$, which must equal the total moment of the internal forces, that is $(m_x + m_x')l_y$. Therefore

$$m_x + m_x' = -\frac{w_0x_1^3l_x^2}{12}[2 - (x_3 + x_4)] \quad . \quad . \quad . \quad (5)$$

Similarly for fragment CDEF, $m_x + m_x'' = +\dfrac{w_0x_2^3l_x^2}{12}[2 - (x_3 + x_4)]$. (6)

The load on an elementary strip b of fragment ADE is

$$-(1 - y_1)x_1^2 x_3 y_1 w_0 l_x l_y . dy_1$$

and, on strip c, $+(1 - y_2)x_2^2 x_3 y_2 w_0 l_x l_y . dy_2$. The total moment about AD on the loads on all strips b from $y_1 = 0$ to $y_1 = 1$ and on all strips c from $y_2 = 0$ to $y_2 = 1$ is $\frac{x_3^2(x_2 - x_1)}{24} w_0 l_x l_y^2$, and must be equal to the total moment of the internal forces $(m_y + m_y')l_x$. Therefore

$$m_y + m_y' = \frac{x_3^2(x_2 - x_1)}{24} w_0 l_y^2 \text{; and } m_y + m_y'' = \frac{x_4^2(x_2 - x_1)}{24} w_0 l_y^2 \quad (7) \ \& \ (8)$$

Example No. 4.—Assume the data for a reinforced concrete retaining wall, freely supported along the two horizontal edges and continuous over the vertical edges, are: l_x (vertical), 13·1 ft.; l_y (horizontal), 16·4 ft.; w_1, 102 lb. per square foot; w_2, 410 lb. per square foot; therefore $w_0 = 410 - 102 = 308$ lb. per square foot, and $w = 0{\cdot}5[102 + 410 + (0{\cdot}55 - 0{\cdot}45)308] = 272$ lb. per square foot.

Assume $x_1 = 0{\cdot}55$, $x_2 = 0{\cdot}45$, and $x_3 = x_4 = 0{\cdot}4$. Due to w, by calculations as in the first part of this article, $m_x + m_x' = 3430$ ft.-lb., $m_x + m_x'' = 2200$ ft.-lb., and $m_y + m_y' = m_y + m_y'' = 2010$ ft.-lb. per foot. Due to w_0, from formula (5), $m_x + m_x' = -0{\cdot}55^3(2 - 0{\cdot}8)308 \times \frac{13{\cdot}1^2}{12} = -880$ ft.-lb.; from formula (6), $m_x + m_x'' = +0{\cdot}45^3 \times 1{\cdot}2 \times 308 \times \frac{13{\cdot}1^2}{12} = +485$ ft.-lb.; from formulæ (7) and (8), $m_y + m_y' = m_y + m_y'' = 0{\cdot}4^2(0{\cdot}45 - 0{\cdot}55)308 \times \frac{16{\cdot}4^2}{24} = -55$ ft.-lb. By addition, and since $m_x' = m_x'' = 0$, $m_x = 3430 - 880 = 2550$ ft.-lb., or $m_x = 2200 + 485 = 2685$ ft.-lb.; the average of these two values is sufficiently correct for design purposes, that is $m_x = 2620$ ft.-lb. Also by addition, $m_y + m_y' = m_y + m_y'' = 2010 - 55 = 1955$ ft.-lb. and, assuming $m_y' = m_y'' =$, say, 860 ft.-lb., $m_y = 1955 - 860 = 1095$ ft.-lb. per foot.

Slab supported on Three Sides: Triangularly-distributed Load.

The common case of a slab, such as the wall of a container, continuous at the bottom edge and at the two vertical edges but unsupported along the top edge, is considered in the following. The intensity of the load is assumed to vary from nothing at the top to w at the bottom. The notation is generally as in the preceding analyses and as in *Figs.* 4 and 5.

Case I.—$l_y > l_x$ (*Fig.* 4).—The lines of probable fracture are DG and CH. The load dP_1 on an elementary strip of the fragment CDGH is $(1 - 2x_3 + 2x_3 y)ywl_y l_x . dy$, and on an elementary strip of the fragment ADG the load dP_2 is $(1 - y)x_3 ywl_y l_x . dy$. If unit displacement at right-angles to the plane of the slab is applied to the edge GH, the fragments remaining plane, the movement of dP_1 is $1 - y$ unit, and the movement of the centroid of dP_2 is $0{\cdot}5(1 - y)$ unit. The work done by the load on the strip of length l_y is $(1 - y)(dP_1 + dP_2)$,

that is $(1 - y)(1 - x_3 + x_3 y) y w l_y l_x . dy$. By integration from $y = 0$ to $y = 1$, the total work A_e done by the external forces is $\frac{2 - x_3}{12} w l_y l_x$. The total moment of the internal forces on the fragment CDGH is $2 m_x x_3 l_y + m_x'' l_y$. Since the angle of rotation of the fragment is $\frac{1}{l_x}$, the work done A_1 by the internal forces is $(m_x'' + 2 x_3 m_x) r$. Similarly, for the fragments ADG and BCH, the total moment of the internal forces about the vertical edges is $[(m_y + m_y') + (m_y + m_y'')] l_x$, the

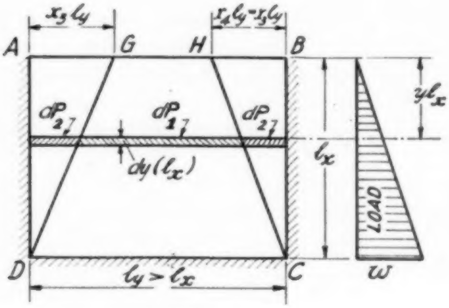


Fig. 4.

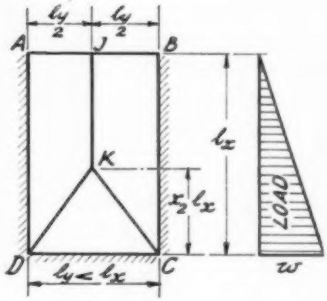


Fig. 5.

angle is $\frac{1}{x_3 l_y}$, and the work done A_2 is $\frac{2}{x_3 r}(m_y + m_y')$. The total work A_i done by the internal forces is $A_1 + A_2$. Since $A_i = A_e$

$$(2 - x_3) w l_y l_x = 12(m_x'' + 2 x_3 m_x) r + \frac{24(m_y + m_y'')}{x_3 r} \quad . \quad . \quad (9)$$

Another equation, which results from the fact that the value of x_3 must be such that the load carrying capacity of the slab is a minimum $\left(\text{that is } \frac{dw}{dx_3} = 0\right)$, is

$$4(1 - x_3)(m_y + m_y') = x_3^2 r^2 (4 m_x + m_x'') \quad . \quad . \quad . \quad (10)$$

If $u_x = \frac{m_x}{w l_x^2}$, $u_1 = \frac{m_x''}{w l_x^2}$, $u_y = \frac{m_y}{w l_y^2}$ and $u_2 = \frac{m_y'}{w l_y^2}$, from (9) and (10)

$$u_1 = \frac{1}{6} - \frac{4}{x_3}(u_y + u_2) \text{ and } u_x = -\frac{1}{24} + \frac{1}{x_3^2}(u_y + u_2) \quad . \quad . \quad (11)$$

The value of x_3 is assumed and also positive values of any two of the coefficients u_y, u_2, u_1, and u_x except u_1 and u_x together. Substitution in (11) gives the values of the other two coefficients.

EXAMPLE No. 5.—The data for a slab supported on three sides are l_y, 13·1 ft.; l_x, 9·8 ft.; w, 308 lb. per square foot. Assume $x_3 = 0{\cdot}3$, $u_1 = 0{\cdot}07$, and

$u_y = 0{\cdot}005$; from the first of formulæ (11), $u_2 = 0{\cdot}0023$. Substituting in the second formula (11), $u_x = 0{\cdot}0393$. Therefore the bending moments are:

$m_x = 0{\cdot}0393 \times 308 \times 9{\cdot}8^2 = 1160$ ft.-lb.; $m_x'' = 0{\cdot}07 \times 308 \times 9{\cdot}8^2 = 2060$ ft.-lb. $m_y = 0{\cdot}005 \times 308 \times 13{\cdot}1^2 = 265$ ft.-lb.; $m_y' = 0{\cdot}0023 \times 308 \times 13{\cdot}1^2 = 120$ ft.-lb. per foot.

CASE II.—$l_y < l_x$ (*Fig.* 5).—The lines of probable fractures are DK, CK, and KJ. By a method of analysis similar to the foregoing, the equation expressing the equality of internal and external work is

$$24(u_x + u_1) + 96x_2(u_y + u_2) - 6x_2 + 4x_2^2 - x_2^3 = f(x_2) = 0, \quad . \quad (12)$$

and from $\dfrac{df(x_2)}{dx_2} = 0$, $u_y + u_2 = \dfrac{6 - 8x_2 + 3x_2^2}{96}$ . . . . . . (13)

Combining (12) and (13), $u_x + u_1 = \dfrac{x_2^2(2 - x_2)}{12}$ . . . . . . (14)

EXAMPLE NO. 6.—The data for a slab supported on three sides are: l_y, 9·8 ft.; l_x, 16·4 ft.; and $w = 510$ lb. per square foot. Assume $x_2 = 0{\cdot}5$, $u_y = u_2$, and $u_x' = 0{\cdot}5u_1$. Therefore $u_x + u_1 = 0{\cdot}031$ from (14), and $u_x = 0{\cdot}0103$ and $u_1 = 0{\cdot}0207$; $u_y + u_2 = 0{\cdot}025$ from (13), and $u_y = u_2 = 0{\cdot}0125$. Therefore the bending moments are:

$m_x = 0{\cdot}0103 \times 510 \times 16{\cdot}4^2 = 1420$ ft.-lb.; $m_x'' = 0{\cdot}0207 \times 510 \times 16{\cdot}4^2 = 2850$ ft.-lb. per foot.

$m_y = m_y' = m_y'' = 0{\cdot}0125 \times 510 \times 9{\cdot}8^2 = 610$ ft.-lb. per foot.

Effect of the Wind on Golden Gate Bridge.

As a result of damage to the Golden Gate bridge at San Francisco, an investigation is to be made on the question of whether the bridge can be strengthened. The bridge is a suspended steel span of 4,200 ft. During a gale in December, when the wind exceeded 50 miles an hour continuously for four hours and reached a maximum of 60 miles an hour, the maximum vertical vibration of the deck was 130-in. double amplitude with a frequency of 8·4 cycles per minute. This occurred at a quarter point, indicating unsymmetrical movement. No conclusions could be reached whether or not there was torsional motion, but eye-witness reports say there was. (It was an unsymmetrical torsional movement caused by the wind that led to the failure of the Tacoma Narrows bridge in 1940.) This was the greatest movement of the bridge that has been recorded. The bridge has been subjected to winds of greater velocity, but none was of such long duration or in a direction to cause such a large movement according to the records of the structure.

Dams in India.

THE Government of the Punjab has engaged Mr. Harvey Slocum, who was the construction superintendent of the Grand Coulee, Friant, Bull Shoals, and other large dams in the United States of America, to supervise the construction of dams in the State. Up to fifty American engineers will accompany Mr. Slocum. The largest dam now in course of construction in the Punjab is the Bhakra dam, which will be 680 ft. high and contain five million cubic yards of concrete. Several other dams are in course of construction or are projected in the Punjab, and it is understood that American engineers are being employed because of the slow rate of progress at present being made.

Book Reviews.

"Reinforced Concrete." By Oscar Faber, C.B.E. (London: E. & F. N. Spon, Ltd. Price 30s.)

THIS book of 228 pages is written in a most interesting manner suited in particular to the novice but it will also be beneficial to experienced engineers. It deals with the subject with particular reference to the British Standard Code of Practice No. 114.

The first chapter traces the properties and uses of concrete from its inception as a structural material and the second deals with progress in recent years, including prestressed concrete. The next two chapters describe the materials of which reinforced concrete is composed as well as their physical characteristics, both separately and in combination, and give valuable information on water-cement ratio, shrinkage, and cube and cylinder strengths. The elementary design of slabs, beams, columns, and footings is clearly dealt with in chapters 5 to 10. Chapters 11 to 17 deal with special applications and structures such as piles, silos, shell roofs, reservoirs, chimneys, precast concrete, and prestressed concrete. These chapters, though short, contain much good information.

The book, in its brief treatment of all these subjects, avoids the use of higher mathematics. In the second chapter attention is drawn to the possible objections to the use in ordinary reinforced concrete of high steel stresses such as 27,000 lb. per square inch and above. The examples of columns, particularly on page 138, illustrate the practical difficulties that occur as the amount of longitudinal reinforcement approaches the 8 per cent. upper limit permitted by the code. The book contains within its short limits much useful information, with many tables and graphs, and is highly recommended.

R. P. M.

"Des Ingenieurs Taschenbuch." Volume III. Bauingenieurwesen. Part 3. (Berlin: Wilhelm Ernst & Sohn. 1951. Price 15 DM.)

THE 27th edition of this handbook deals with foundations and earthworks; waterways; dams and hydro-electric works; and water supply, sewerage, and sewage disposal. The basic problems and theories are stated concisely, and there are many excellent illustrations and copious references. Foundations in ground and under water are described in great detail. Data are given for any combination of surcharge against retaining walls, with diagrams relating to retaining walls, quay walls, and abutments. Foundations for machines are discussed, with special reference to preventing vibration. Equations are given for determining economical haulage distances in tunnels. In the section on waterways are discussed hydrological principles, planning and design of canals, channels, river improvements, estuaries and tidal work, docks and harbours, and land drainage. Considerable attention is given to the design of dams of the arch type and the economics of hydro-electric works. The notes on the disposal of storm-water are interesting insofar as empirical formulæ still appear to be used, whereas in this country such calculations are based on the Lloyd-Davies method of 1906 with improvements such as the time-area graph.

"The Structural Analysis of the Dome of Discovery." By T. O. Lazarides. (London: Crosby Lockwood & Son, Ltd. Price 25s.)

THE original design of the Dome of Discovery at the Festival of Britain was an approximate treatment. An exact analysis was, however, later carried out by the method of relaxation for statically indeterminate structures as described by Professor R. V. Southwell. The author gives a complete description of the method and details of the calculations, including the loading assumptions, the choice of the axes, the calculation of the influence coefficients, the preparation of the conversion, operational and main relaxation tables, and the recompounding of the individual members. A full discussion of the behaviour of the structure under load completes this record of redundant space-frame calculation.

The Pathology of Reinforced Concrete.

By HENRY LOSSIER.

THE following is an abstract of a book entitled " La Pathologie du Béton Armé," written by M. Henry Lossier, the eminent French engineer, and published by Editions Dunod, of Paris, which is translated here by arrangement with the author and publisher. In this abstract, which will be continued in later numbers of this journal, M. Lossier gives much valuable information based on his long experience. The translation from the French is by Mr. R. E. Walsh, M.A., B.Sc.

CHOICE OF TYPE OF STRUCTURE.

Curved Beams, or " Flat Arches ".

A special case is the flat " arch ", that is a beam with a straight extrados and a slightly arched intrados. The design of these beams was the subject several years ago of many discussions ; some engineers treated them as fixed-ended beams of variable section and others as true arches subjected to horizontal thrust. This distinction is, if not theoretically at least practically, of no moment. If it is assumed that the deflection of the intrados is comparatively great, and if the stresses are computed on the basis of an elastic arch fixed at its ends—that is to say, assuming that the supports can resist a horizontal thrust without deforming—a line of thrust inclined towards the supports may be drawn approximately as shown in *Fig.* 1. As the deflection of the intrados diminishes the thrusts R become more nearly vertical and the horizontal thrust diminishes. On reaching the limiting case of zero deflection, the two thrusts are then wholly vertical and there is no horizontal thrust. There is thus no sudden change from the functioning as an arch to the functioning as a beam. In the case of very flat members used in structures of this kind, the results of the two methods differ only slightly.

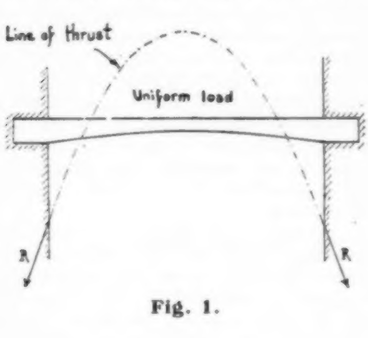


Fig. 1.

Fig. 2.

The foregoing naturally assumes a normally reinforced member. If, as has sometimes happened, the reinforcement is incapable of resisting the imposed bending moments, the beam functions in the manner represented by *Fig.* 2. At first it behaves as an arch with fixed ends and is subjected to large bending moments at the supports and at mid-span ; but these sections, being insufficiently reinforced, crack and the member opens in the extrados at the supports and in the intrados at mid-span. As a consequence of this self-adjustment, the member becomes a true arch which either stands or falls depending on whether or not the supports are capable of resisting the horizontal thrust without yielding. Since it is exceptional for the supports to be capable of resisting large horizontal thrusts without yielding, it is generally prudent to regard spans of this kind as beams of variable section with ends either fixed or freely supported according to the conditions.

Series of Arches.

A series of arches of equal span without tie-bars, with walls or rocker bearings as intermediate supports, and capable of resisting only purely vertical forces, was the cause of one of the worst accidents recorded in the history of reinforced concrete. Arches of this kind, which had to

carry a load of earth filling, were used to roof a large reservoir. Each span had been calculated as an arch on the assumption that, as the spans were identical and equally loaded, the horizontal thrusts would balance one another at each intermediate support. On covering the roof, instead of spreading the filling uniformly over all the spans, the earth was placed on a few spans only. The horizontal thrust from the loaded vaults, not being balanced by that of the neighbouring unloaded ones, caused the former to flatten with accompanying complete collapse of the whole structure as represented in *Fig.* 3.

In order that multiple vaults of this kind should be stable, they must comply with the two conditions depicted in *Fig.* 4; first, the outer supports A and B must resist without yielding the inclined thrust *R*, and secondly the load must be uniformly applied over all the vaults.

Generally, accidents of this nature are attributable to one of two causes, namely, the designer is content to provide ties in the outer spans, without bothering about the strength and rigidity of the end supports; or the loads on the different spans are not kept equal. When a series of vaults must support variable loads the stresses can be determined with precision by the method of analysing a continuous elastic arch. When structures of little importance, which do not justify such an exact analysis, are concerned, it is sufficient to consider each span as an arch with fixed supports under the action of the dead load, and as a continuous beam, integral with its piers, under the action of the live load.

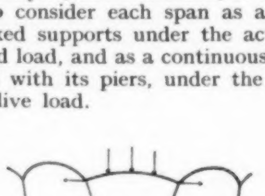


Fig. 3.

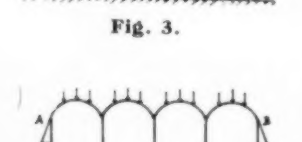


Fig. 4.

Continuous Beams.

Some engineers think that continuous beams should be designed as homogeneous beams, that is according to the common formulæ based on the elastic theory. Others consider that they should be designed by special empirical methods, assuming only partial continuity. Beams designed and constructed for perfect continuity exhibit, in general, the advantage of cracking less than those designed for partial continuity. In effect, because the reinforcement is designed to resist tensile forces both at the supports and at intermediate sections, the limits of fatigue normal to the steel and the concrete are not exceeded anywhere, unless it be to an insignificant extent.

Partially-continuous beams have often been designed by adopting the simple formula $\frac{wl^2}{10}$ for the bending moment between the supports, half only of the bars being bent-up over the supports. Under the action of the dead load, the stresses in sections over the intermediate supports tend to be higher than those for which the reinforcement is designed. Due to the resulting excessive deformation the distribution of the bending moments adjusts itself, the sections at the supports which are too weak relaxing at the expense of the stronger sections between the supports. There is then formed (sometimes at the cost of cracking of the extrados over the supports) a new state of equilibrium which tends to approach automatically the original hypothesis without any real sacrifice of safety. The facility possessed by statically indeterminate structures (as also, moreover, by living organisms) of adapting themselves to abnormal conditions, with the object of delaying as much as possible the moment of their dissolution, is an almost universal phenomenon.

In practice, a beam designed with monolithic spans on the assumption of partial continuity does not represent, in principle, a pathological case, provided that (*a*) the relative weakness of the sections over the supports is compensated by a corresponding excess of strength in the intermediate sections, and (*b*) the difference between the allowable values for the moments and those considered as limiting remains within the range usually

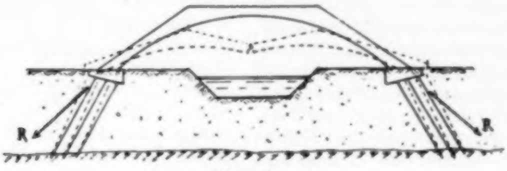


Fig. 5.

accepted. A few cracks causing no risk to the stability of the structure are normally the only objection to this departure from the common theory.

Arches and Vaults.

The arch bridge is a type widely built in reinforced concrete because it is in general the most economical.

The adoption of a hingeless hyperstatic arch, when the abutments are not absolutely rigid, is always an erroneous conception. It is, however, only seldom that a true failure occurs, since the arch forms its own hinges on cracking, but more or less serious damage may be caused. The use of temporary hinges or of lowering the centering by jacks is only a partial remedy, because eventual shrinkage and settlement of the abutments can sometimes continue for several years before they become quite stable. One of the most typical cases is a footbridge of 140-ft. span, built over a canal in the year 1931 (*Fig.* 5). This bridge was founded on gently-raking piles bearing on rock underlying mud of no consistency. The inclination of the thrust *R* of the bridge not coinciding with that of the piles, and the resistance of the mud being incapable of preventing movement of the piles, the abutments spread, following the removal of the centering of the arch, with increasing rapidity leading to the complete collapse of the bridge within six days. It is essential in such cases either to construct the foundations of stable monolithic material or to provide abutments of piles having varying slopes according to the thrusts of the arch as generated by all possible combinations of loading.

CALCULATIONS OF STRENGTH.

Theory and Reality.

It is possible to be at one and the same time an outstanding mathematician and a mediocre engineer, for in engineering, as in many other arts, pure science is of little avail unless it is directed by sound common sense. Two principal considerations must be borne in mind : First reality, which is independent of any ideology or preconceptions ; Secondly theory, which is of human origin and of arbitrary character, which seeks to impart a simplified and comprehensible image of reality, enabling reasoning and the formulation of conclusions to be made with the least risk of error. No theory coincides exactly with reality, and their divergencies are the principal cause of most of the false reasoning of theorists. Who has not, in identifying an approximate image (which is the theory) with the object itself (which is the reality), followed indefinitely the thread of his reasoning oblivious of the fact that he has long before left the track of healthy logic ? In metal construction the working of most structures depends on a smaller number of factors than does reinforced concrete structures, because the latter are subjected to the factors of monolithism and variations with the weather of the characteristics of some of their components. This is why builders in reinforced concrete are particularly exposed to the vices of impenitent theorists.

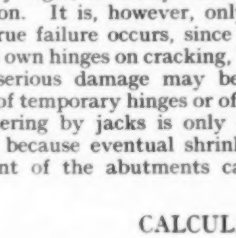


Fig. 6.

SLAB-AND-BEAM CONSTRUCTION.—Consider as an example the case of a simple slab with parallel and equidistant beams (*Fig.* 6). In designing this slab the common formulæ applicable to a continuous beam of uniform section resting freely on unyielding supports are most frequently used. Yet (*a*) the slab is not of uniform section since it has two layers of reinforcement over the beams and a single layer between them; (*b*) the slab does not rest freely on its supports since it is integral with the beams which resist the torsion; and (*c*) the supports of the slab are not unyielding since the beams deflect unequally under the action of the varying live loads; this deflection diminishes towards the ends. The striking differences which exist between the assumptions made by designers and the reality in this reputedly simple case demonstrate how illusory may be the results of some accepted methods of calculation.

BOW-STRING BRIDGES.—As a second example, consider the case of a bow-string bridge (*Fig.* 7) comprising a lightly-reinforced concrete arch and a horizontal tension member having a high percentage of reinforcement. In course of time the shrinkage and creep of the concrete have a much greater effect on the linear variations of the arch than on those of the tie. Also the functioning of the structure will change, under both the live and dead loads, after its completion, and for important bridges an attempt should be made to evaluate the extent of these changes.

MULTIPLE-SPAN BRIDGES. — Arch bridges comprising multiple spans have also suffered from bad theory. In the early days of my career I designed a reinforced concrete viaduct with arches of equal span carried on high piers. Using the theory of an elastic ellipse I took into account the deformation of both piers and arches due to accidental loads. Compared with the approximate method, based on the assumption of a series of arches on fixed supports in every case, this method, undoubtedly more exacting, led logically to a thickening of the arches and a thinning of the piers. This design was submitted in a competition, and the assessor was fiercely opposed to what he called a heresy. After a demonstration of principle with a model made of easily deformable material, he asserted that he would never agree that a test on a model could challenge a method he had taught for many years.

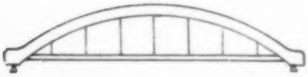


Fig. 7.

THE MODULAR RATIO.—The coefficient m, by which the area of steel must be multiplied to make it equivalent to the area of concrete, has been the subject of much controversy. There were originally two opposing doctrines. Hennebique proposed an arbitrary compressive working stress in the concrete of 355 lb. per square inch and of 17,000 lb. per square inch in the steel, thus giving on m a value of 48. The orthodox school, starting with the premise that concrete has a mean coefficient of elasticity of about one-tenth of that of steel, decided that $m = 10$, and this value was adopted in many regulations. If this value has on occasions led to some needless extravagance in reinforcement, it has never been the cause of a failure.

Later, in order to take into account several factors, and in particular the effect of the concrete in tension on the position of the neutral axis in members subjected to bending, a modular ratio of 15 was adopted without regard to the formula derived from actual tests by M. Caquot and which logically incorporated the percentage of reinforcement. This constant value of 15 is much used in current practice, but I have often wondered why so many engineers have such an affection for the modular ratio. Let us follow the normal history of a compression member in service. At the outset of its life it will probably act as though the steel were equivalent to the concrete, but with its section multiplied by $m = \frac{E_s}{E_c}$, E_s and E_c being the respective coefficients of elasticity of the two materials. Then, under the combined effect of shrinkage during hardening and creep, the concrete, by gradually shortening, will free itself partially from its initial restraint, thereby transferring load to the steel which has not shortened. In other words, the coefficient m will pass from a value ranging from 7 to 10 to a value which might exceed in certain cases three times these values.

It appears to be illogical to use a fixed

value of *m* which can agree only for the time being with reality, and which affects the cost of construction only very slightly except in the case of compression reinforcement, the use of which is gradually being abandoned. Let a modular ratio of 10, 15, or even 20 be used, but do not let an academic viewpoint prevail when it is not in accordance with reality.

STRENGTH OF CONCRETE.—A few words on the theoretical exactness of calculations of the strength of concrete. There are some who persist in working to an imposing number of decimal places, although the practical agreement between the theories and reality is not greater than 70 per cent. to 80 per cent. There are others who, with greater justification, repose unlimited confidence in calculations based on the mathematical theory of elasticity. Again, basic hypotheses may differ very greatly from reality. On the one hand reinforced concrete is not in every respect a homogeneous material, and on the other hand the errors of the hypotheses are multiplied along with the normal factors. In short, the functioning of reinforced concrete structures can seldom be defined theoretically with great exactitude even by the most orthodox calculations of strength.

Hennibique carried out a number of works by applying a method sometimes considered, not without reason, to be heretical since it leads to the equating of an external bending moment with a moment of resistance formed by two forces, one tensile and the other compressive, which are not equal. Nevertheless no failure has occurred as a result of the use of this method, thanks to the sound common sense of its author and, it must be admitted, to the phenomenon of adaptation which compensates in some cases for the designer's errors. An engineer worthy of the name ought to have sufficient mathematical knowledge to enable him to comprehend all the current theoretical difficulties; but this knowledge will be of little value unless he possesses, in addition, the rare quality of common sense.

(*To be continued.*)

Precast Concrete Wall Slabs.

As the result of experience of the use of precast concrete insulated wall slabs on a power station building, on which 60,000 sq. ft. of such panels were erected a year previously, the slab shown in *Figs.* 1 and 2 has been adopted by the Union Carbide and Carbon Corporation of the U.S.A. for 500,000 sq. ft. of walls at a new plant at Marietta, Ohio.

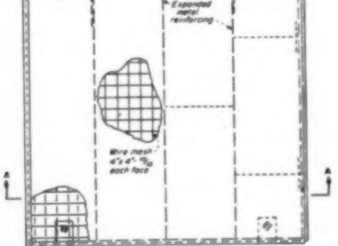


Fig. 1.

The area of the slabs, which are in two sizes measuring 8 ft. by 8 ft. and 8 ft. by 10 ft., is a compromise between the desire to have as few joints as possible and ease of handling the slabs. The height of the slabs also conforms to the window heights and other features of the elevations.

The slabs are 5 in. thick, and comprise two outer layers of concrete $1\frac{3}{4}$ in. thick and a core of insulating material $1\frac{1}{2}$ in. thick. The concrete has a compressive strength at 28 days of 4000 lb. per square inch, and comprises aggregate of $\frac{3}{4}$ in. maximum size and 560 lb. of high-early-strength Portland cement per cubic yard. Each layer of concrete is reinforced with 4-in. square mesh supplied in flat sheets. The two layers of concrete are joined by strips of expanded metal bent channel shape, as shown in the section in *Fig.* 1. Two U-shape bars project from the top

of the slab for use as lifting hooks, and threaded inserts to receive connecting bolts are embedded near the top and bottom of each slab.

As 3000 sq. ft. of slabs are being made daily, four different insulating materials are used in order to be sure of continuity of supply. These are (1) precast wood-wool slabs, (2) precast slabs made with wood chips, (3) cellular glass, and (4) glass-fibre. These are supplied in sheets 2 ft. wide by 8 ft. long. The coefficient of heat transfer of these materials ranges from 0·14 to 0·33 B.Th.U. per hour per square foot per degree Fahr., compared with 0·36 for a 12-in. brick wall.

Tongue-and-groove joints are formed on all four edges. Before the slabs are erected a continuous strip of impervious cellular rubber is cemented in the grooves, and after the slabs are erected the joints are packed with oakum and caulked. The edges are slightly bevelled to reduce damage to the arrises, and also to make less noticeable any slight irregularities.

The slabs are made in steel moulds. The inner face has a matt finish produced by using muslin in the bottom of the mould. The outer face is finished with a wooden float and brushed to produce uneven shallow grooves vertically.

At every third course the slabs are supported on the steel frame of the building by means of clips as shown in *Fig.* 2. The slabs are hoisted by a crane, and 2000 sq. ft. are normally erected in an 8-hour day by a foreman and ten men. The foregoing notes and the illustrations are from "Engineering News-Record" for January 24, 1952.

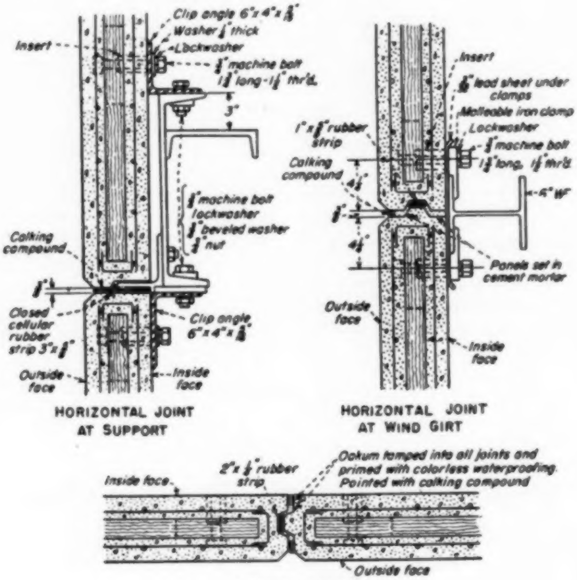


Fig. 2.

Analysis of Groups of Piles.

A Semi-Graphical Method Based upon Displacements.

By R. J. WILKINS, M.Sc.(Eng.), A.M.Inst.C.E., M.I.Struct.E.

THE January, 1952, number of "Concrete and Constructional Engineering" contained an article by Mr. G. P. Manning on the analysis of groups of piles by the displacement method in which a general treatment was developed for the case of two inclined piles joined at the apex. This problem lends itself to a convenient semi-graphical method which may be applied directly to any numerical case and which forms an interesting extension of the methods previously described.

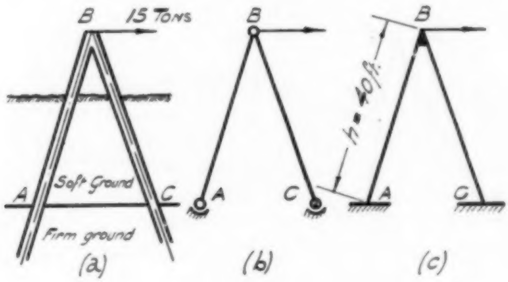


Fig. 1.

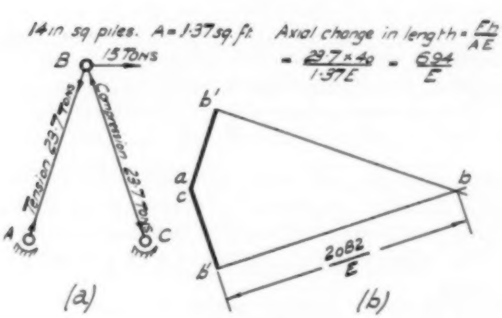
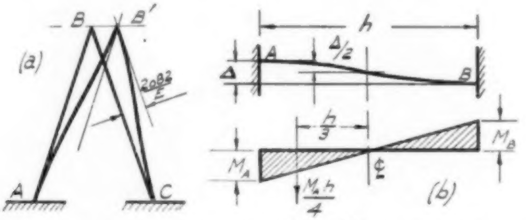


Fig. 2.

No new theoretical treatment is involved by the semi-graphical method, which is based on the common methods used in moment-distribution analyses for the determination of secondary bending moments in framed girders with rigid joints. For comparison, the numerical examples used in Mr. Manning's article are used to illustrate the method, and the same notation is employed. The selection of any particular method of structural analysis is largely a matter of personal preference, and claims of superiority must be carefully assessed. The present method may appeal to some designers because of its numerical and graphical basis.

Fig. 1 shows two identical raking piles subjected to a horizontal pull of 15 tons. As pointed out by Mr. Manning, such piles are usually designed as pin-jointed at both ends as in *Fig.* 1*b*, although actually they may be monolithic with the super-

structure at the apex B and approach fixed-end conditions at A and C as shown in *Fig.* 1*c*. Considering the pin-jointed structure in *Fig.* 2*a*, the forces in the piles will be 23·7 tons direct tension in pile AB and direct compression in pile BC. The piles have a slope of three vertically to one horizontally. These forces will cause axial changes of length of $\frac{694}{E}$ in each case. A Williot displacement diagram may now be drawn for the pin-jointed structure as in *Fig.* 2*b*, in which ab′ represents to scale the lengthening of AB, and cb′ the shortening of BC. Lines drawn normal to the direction of the axial changes in length represent the arcs of rotation about A and C and enable the displaced position of B to be determined from the intersection of the lines. *Fig.* 3*a* shows the displaced position of the fixed-ended



By moment-area theory: $\frac{\Delta}{2} = \left(\frac{M_A h}{4EI}\right)\left(\frac{h}{3}\right)$; $M_A = M_B = \frac{6EI\Delta}{h^2}$

$$M_A = M_B = \frac{6E(1{\cdot}17)^4\, 2082}{40 \times 40 \times 12 \times E} = {\cdot}121 \text{ ton ft.} = {\cdot}32{,}400 \text{ lb. in.}$$

A	B	B	C	
	0·5	0·5		Distribution Factors
−32,400	−32,400	−32,400	−32,400	
+16,200 ←	+32,400	+32,400 →	+16,200	
−16,200	0	0	−16,200	Final Moments

Fig. 3.

structure in which the members AB′ and B′C are considered to be held by external restraining moments in such a manner that the tangents at the extremities are parallel to the original position of the member but displaced from it by a relative displacement Δ. The value of this displacement may be scaled from the displacement diagram as $\frac{2082}{E}$.

When a member of uniform EI is fixed at its ends and these ends are then displaced a relative distance Δ, but remain parallel, then moments are produced at each end of the member of magnitude $\frac{6EI\Delta}{h^2}$. The proof of this formula is given in text-books; it is one of the basic formulæ in moment-distribution methods. *Fig.* 3*b* shows how the formula is derived from the moment-area properties of the elastic curve. Substitution of $\frac{2082}{E}$ for Δ, and the values for I and h, give the fixing moment at each end of AB and BC as 32,400 in.-lb. while the members are externally restrained in position. Note that a numerical value of E is not required since it cancels out upon substitution of Δ.

In the actual structure, for the conditions shown joint B is free to rotate

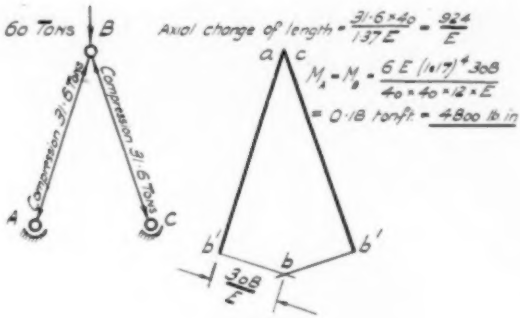


Fig. 4.

and no external moment is applied at this joint. *Fig.* 3 shows the moment-distribution table for the one operation of releasing joint B and giving the final moment $M_A = M_C = 16{,}200$ in.-lb. as previously obtained. *Fig.* 4 shows the semi-graphical method applied to the same structure when subjected to a vertical load of 60 tons applied at B. In this case no balancing operation is required, and $M_A = M_B = M_C = 4800$ in.-lb. as before. The unbalanced shearing forces which arise from this method are very small and may be neglected. Cases of unsymmetrical piles, different end conditions, and so on, may be allowed for by the common methods of moment distribution when extending the analysis to other cases.

The valuable comments made by Mr. Manning on the effect of ground conditions, the effective length of the pile, and the influence of hard layers near ground level remain valid, since the assumptions made for a particular design depend upon engineering judgment and in themselves are just as important as the structural analysis. They are clearly independent of the particular method employed for a structural solution.

A Bridge-testing Machine.

A VEHICLE weighing 85 tons for testing the strength of road bridges has been completed for the Ministry of Transport. The main feature of the machine is a slab of concrete 35 ft. long by 7 ft. 9 in. wide by 3 ft. thick and weighing 50 tons. This slab is mounted on rails on a solid-tyred well-deck trailer in such a manner that it can be rolled along until nearly half its length overhangs the end of the trailer. A further load of 30 tons of ballast can be piled into a well at the top of the slab, and the weight on one bogey can be varied from 20 to 90 tons. Tests on road bridges were to begin in June. The machine is to be used mainly to test the ability of masonry arch bridges to carry heavy loads.

A Precast Reinforced Concrete Dome.

THE construction with precast reinforced concrete segments of a dome of 100 ft. diameter in a church recently rebuilt in Karlsruhe is described in a recent number of " Beton-und Stahlbetonbau." The dome comprises sixty-four identical segments, the bases of which are supported on a tension ring on masonry walls, and the tops against a thrust ring which also supports a skylight. Provision is made for a false domed ceiling to be attached later. Timber scaffolding (*Fig.* 2) erected to support the thrust ring also supported the pivot of a rotating gantry crane (*Fig.* 3), the other support of which was a bogie on a rail on the tension ring. The segments were cast on the ground floor of the structure (*Fig.* 1) and placed in position by the crane. To avoid unbalanced thrusts on the thrust ring during construction, the segments were erected in pairs opposite one another (*Fig.* 2). When all the segments had been erected, the joints in the thrust and tension rings were filled with concrete.

The segments act as curved ribs, there being no circumferential stresses owing to the joints between the segments. Circumferential forces are resisted entirely by the thrust and tension rings. Under the most unfavourable loads, the stresses in the segments are almost entirely compressive. Only during erection were the segments subjected to large bending stresses, when they had to support their own weight and resist wind forces while acting as freely-supported beams. In this temporary condition the tensile stress in the reinforcement was 28,500 lb. per square inch.

The segments are tee-shape, the thickness of the flanges being 2 in. They are of dense concrete, so it was not considered necessary to provide a waterproof covering except at the cornice. At the abutting edges of the segments the thickness of the flanges is 4 in. and a watertight joint is formed by a covering of semicircular concrete tiles. The ribs of the segments are 16 in. deep and taper generally from 4 in. at the bottom to about 7 in. at the junction with the flange, thus permitting easy removal from the moulds. The radius of the segments is about 61 ft.

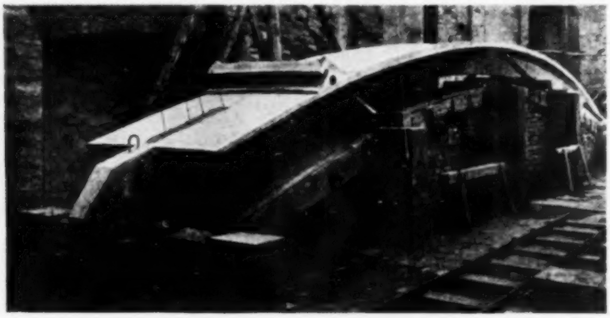


Fig. 1.—Mould for Making Segments.

The curved parts of the moulds (*Fig.* 1) were faced with concrete. A rough mould was made first from old bricks and masonry and then finished accurately with reinforced concrete. Three moulds were employed, each being used more than twenty times. No special measures were used to accelerate the hardening of the concrete. The concrete surfaces of the mould formed a smooth finish on the segments. After being lifted vertically from the mould each segment, which weighed about 4 tons, was turned lengthwise and hoisted obliquely, and for this operation two hand-winches were used in conjunction with the crane. The total weight of the reinforced concrete dome is much less than the weight of the former wooden dome.

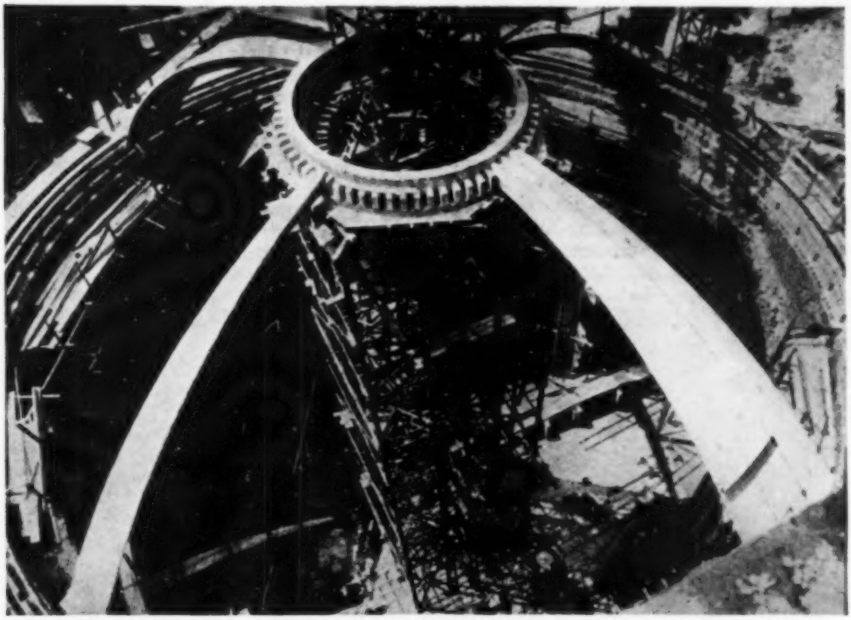


Fig. 2.

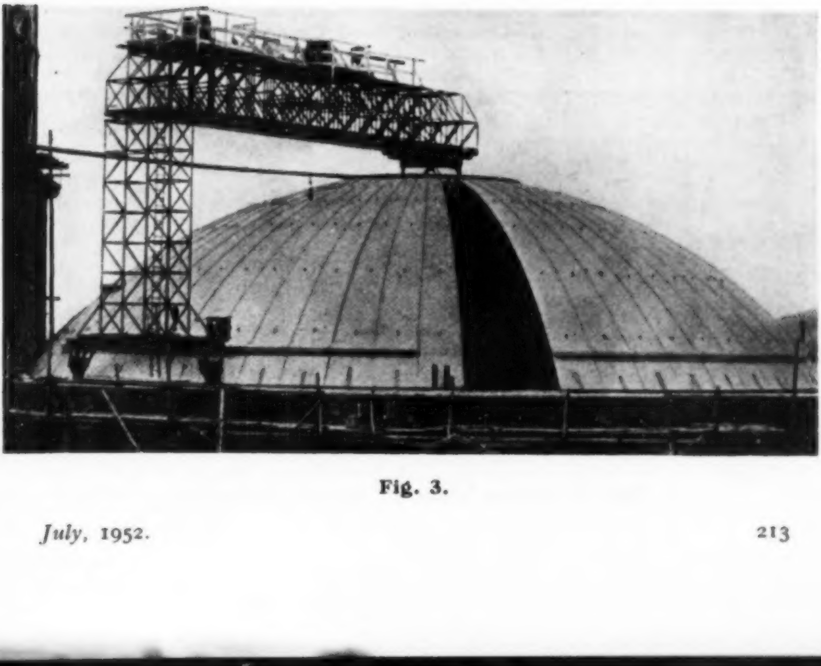


Fig. 3.

Mobile Concreting Plant.

THE illustrations show the method of concreting channels on the main road near Doncaster by the use of a mobile plant. The work was done for the West Riding of Yorkshire County Council under the direction of Mr. C. Maynard Lovell, O.B.E., County Engineer.

The mixing plant consisted of a weigh-batcher, a high-lift loading shovel, and two 10/7 concrete mixers. The latter were raised to discharge directly into the lorries. Three lorries (*Fig.* 1) were provided with an opening in the side; this was fitted with a sliding door on the inside of the lorry and a detachable chute was made to enable the concrete to be placed directly into position.

The channel was provided with a thin surface course of white concrete. To mix and place this material a mobile plant

Fig. 1.—Concreting Road Channel.

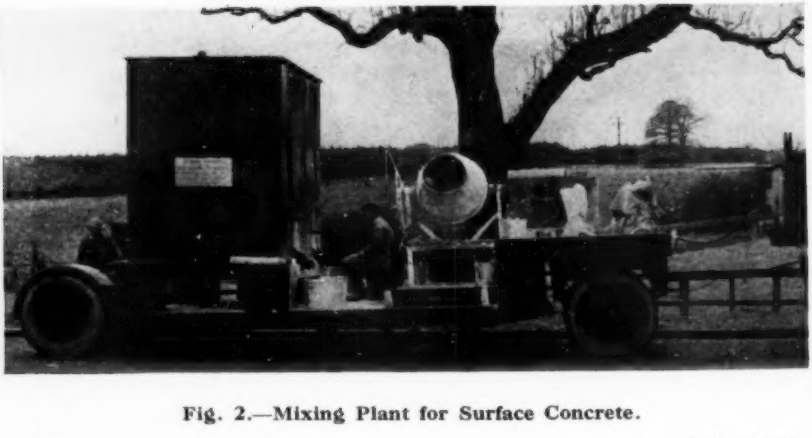


Fig. 2.—Mixing Plant for Surface Concrete.

mounted on a 5-ton low-loader trailer (*Fig.* 2) was set up consisting of storage hoppers for ⅜-in. aggregate and limestone dust with controlled outlet chutes, and a 5/3½ concrete mixer discharging into a hopper provided with a chute to a controlled discharge outlet. The trailer was towed by a lorry carrying a water-storage tank from which water was fed by gravity to the mixer. White Portland cement for the topping was also carried in the lorry.

By the use of these lorries, the mobile mixing plant, and a mechanical ditcher for excavation, a maximum output of 450 linear yards per day was obtained.

The Reinforced Concrete Association.

THE supply of reinforcement bars was the main topic discussed at the annual luncheon of the Reinforced Concrete Association held in London last month.

Mr. J. Cuerel (the President) said that, although the Government was convinced that reinforced concrete was a steel saver rather than a steel user, the supply of reinforcement bars was not sufficient for requirements. In the year 1950 the consumption of reinforcement bars was 200,000 tons, in 1951 it was 240,000 tons, and for the months of 1952 for which figures were available the consumption was at the rate of only 200,000 tons a year, of which about one-quarter was imported. The consumption of reinforcement bars was the criterion of the progress of reinforced concrete, and it was obvious that the allocation of steel for reinforcement bars was insufficient for the nation's needs. In the United States of America the consumption of reinforcement bars per head of population was twice as much as in Great Britain. The present scarcity resulted in designing to suit the sizes of bars available, work was delayed because the contractor had to wait for bars to be delivered, and men did not work well if they did not know when the next supply of bars would arrive. The speaker pleaded for a larger allocation of reinforcement bars so that full advantage could be taken of the economy in steel that resulted from the use of reinforced concrete. There was now a better supply of wire for prestressed concrete.

The President mentioned that the Association was increasing its membership, and that it was influential in the preparation of codes of practice, by-laws, and other regulations relating to reinforced concrete. The Association was considering the formation of a new class of membership for works supervisors and men in similar positions.

The Minister of Works (The Rt. Hon. David Eccles, M.P.) said that he was convinced of the great economy of steel made possible by the use of reinforced and prestressed concrete, and had instructed his Ministry to use these materials wherever possible. He was also urging the Ministry of Supply to make available larger quantities of bars and wire. As soon as circumstances permitted he would abolish as many as possible of the present restrictions and duties of the Ministry of Works, so that the industry could again manage its own affairs.

The Council for the ensuing year is as follows: President, Mr. M. G. Cowlishaw (Guest, Keen & Nettlefolds (South Wales), Ltd.).

Vice-President, Mr. H. G. Cousins.

Members: Messrs. E. Bellringer (Stothert & Pitt, Ltd.), E. J. Cook (Richard Costain, Ltd.), E. L. V. Dakin, M.C. (Wilson Lovatt & Sons, Ltd.), E. R. Hole (Holland & Hannen and Cubitts, Ltd.), C. H. S. Howkins, O.B.E. (Howard Farrow, Ltd.), W. K. Laing, M.A. (John Laing & Son, Ltd.), L. J. Murdock, Ph.D., M.Sc. (George Wimpey & Co., Ltd.), H. F. Rosevear (Sir Robert McAlpine & Sons, Ltd.), E. S. Shellard (Associated Portland Cement Manufacturers, Ltd.), H. E. Steinberg (Considère Constructions, Ltd.), G. W. Stokes (Matthews & Mumby, Ltd.), F. W. Sully (Holloway Bros. (London), Ltd.), J. W. Vanderbeeck (British Reinforced Concrete Engineering Co., Ltd.), and R. W. Vawdrey (Indented Bar & Concrete Engineering Co., Ltd.).

Associates: Messrs. P. G. Bowie, S. Champion, Ph.D., M.Sc., F. C. Eales, J. R. Liversedge, D. H. New, J. North, and F. S. Snow.

Branch Delegate: F. L. Abel (Richard Abel & Sons, Ltd.).

Past Presidents: Messrs. J. Cuerel, E. Wingrove Keer, A. R. Neelands, M.C., J. Singleton-Green, M.Sc.

Foundation for a Turntable.

The illustrations show a prestressed concrete foundation for a turntable recently built at Wath-on-Dearne in connection with the electrification of the Manchester-Sheffield-Wath railway. Prestressed concrete was used because the site is liable to mining subsidence, and settlements up to 4 ft. have taken place nearby. The design was produced in the department of the Civil Engineer of the Eastern Region of British Railways.

The foundation is in the form of a circular rim of 70 ft. diameter with four main radial beams and four secondary radial beams. The rim is 3 ft. 6 in. high by 2 ft. wide. The radial beams are 3 ft. 6 in. high by 1 ft. 6 in. wide. The prestressing wires for the rim are in two chases on the outside of the rim; there are two cables each of 24 wires in each chase. In the radial beams there is a cable comprising 64 wires near the top and a cable of 56 wires near the bottom. The rim and the radial beams are designed to act either as beams or cantilevers, so that in the event of local settlement the foundation will be capable of carrying an engine without breaking and be restored to its former level by jacks. The anchorages for the wires are provided by Magnel-Blaton plates. Mr. C. R. Price was the contractor.

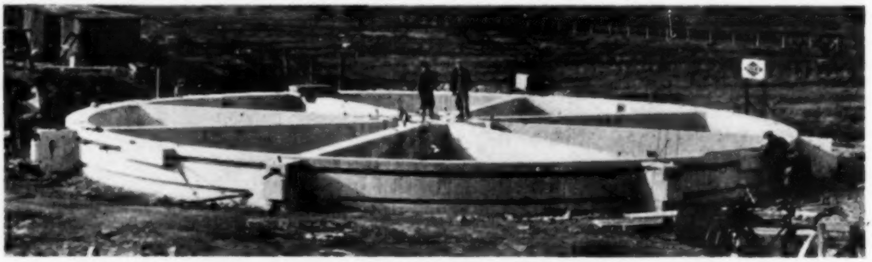


Fig. 1.—Foundation During Construction.

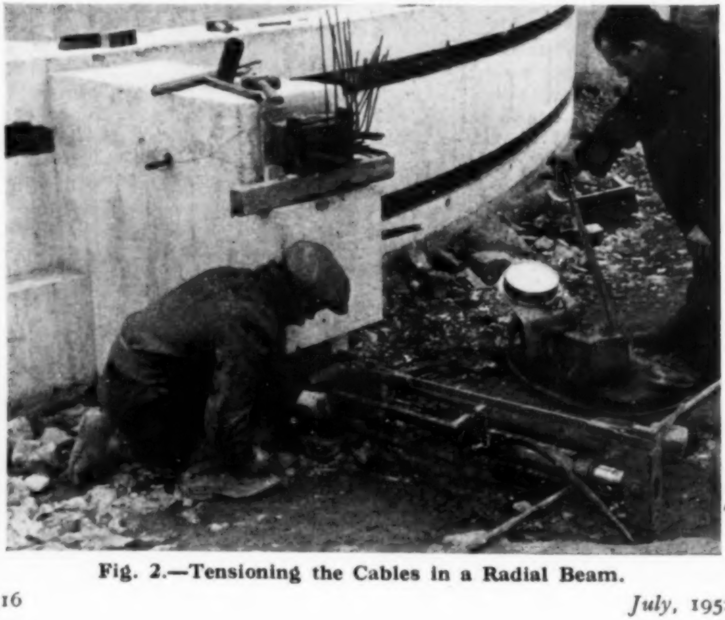


Fig. 2.—Tensioning the Cables in a Radial Beam.

Concrete Piles in Holland.

By P. F. van der MEULEN BOSMA.

DUE to the nature of the soil, large numbers of piles are used in the Netherlands. When they are made on the site the piles are generally cast on thin concrete bases. To economise in moulds and space the following practice is commonly used : First piles Nos. 1, 3, 5, etc., of a layer are made and are exactly spaced to permit other piles to be made between them. When the moulds are taken away, the piles already made are covered with paper or coated with clay and piles Nos. 2, 4, 6, etc., are cast between them. This procedure is repeated for succeeding layers. Concrete piles up to 80 ft. long with a diameter of 16 in. and weighing up to 8½ tons are made in factories by the centrifugal process. Generally, however, factory-made piles are seldom longer than 30 ft. due to the difficulty of transport.

Enlarged Points.

In addition to normal types of points, a point with an enlarged cross section (*Figs.* 1 and 2) and a patented " wing " point (*Fig.* 3) are used in soils where frictional resistance is practically nil but under which a stiff layer is found. The points shown in *Figs.* 1 and 2 are similar to the points of large piles, but to save weight and material the enlarged section ends above the point and is replaced by a smaller cross section ; for example, if the enlarged section is 16 in. square the upper part of the pile is 12 in. square. The increase of the load-carrying capacity of the point is then about 80 per cent. The " wing " point, due to its larger contact area with the soil, can carry a load about 50 per cent. greater than a normal point.

Piles with normal points are often made without pallets on an oiled concrete base. Piles with special points are made on pallets. The moulds are in most cases fixed with joiner's cramps. In some cases the longitudinal bars are bent over just under the top, so the ends of the bars can be exposed easily when the piles are driven.

The city of Rotterdam has patented a special method of casting piles with enlarged points (*Fig.* 4). The area of the cross section of the point is twice the area of the remainder of the pile. Piles No. 1 are first cast. After removal of the moulds, piles No. 2 are cast between piles No. 1. This system is very economical in the use of moulds and space. The piles are marked with painted bands before they are moved ; black bands indicate the places where the piles are to be supported when they are lifted horizontally, stacked, or transported, and red

Fig. 1.

Fig. 2. **Fig. 3.**

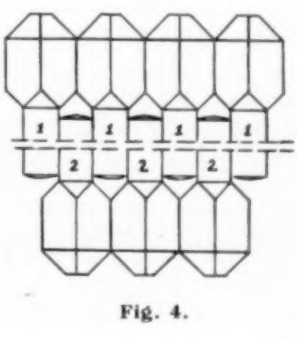


Fig. 4.

bands show the places where the piles are to be supported when they are lifted for driving.

Piles in Two Parts.

A patented pile, known as the "Duplex," is shown in *Figs.* 5 and 6. These are made in total lengths up to 70 ft. with diameters of 10 in. and 12 in. The parts are made in maximum lengths of 35 ft. The top of the lower part has a circular section 1 ft. 4 in. long. The lower end of the upper part has an enlarged section 1 ft. 4 in. long which is lined with four oak guide-strips. When the lower part is partly driven mortar is placed on the top of it, so that when the upper part is placed on it and driven the mortar is pressed into the joint; a ring of rope, placed in a groove around the lower edge of the upper part, prevents loss of mortar. The advantages of this type of pile are that the shorter pieces are easier to transport, lift, and drive, and that a shorter and lighter pile-driver can be used. The joint is claimed to be as strong as a normal section.

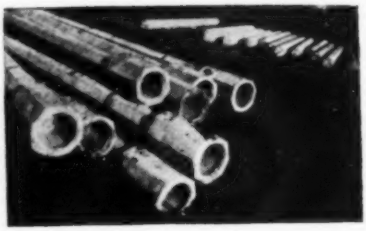


Fig. 5.

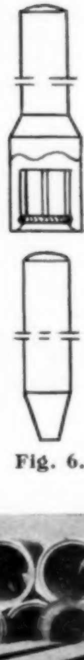


Fig. 6.

Sheet Piles.

Sheet piles are extensively used in the Netherlands. Generally these are the common type with V or tongue-and-groove joints.

Sheet piles with semi-circular grooves in the sides (*Fig.* 7) are used to line canals in soils containing much fine sand. They are placed with a water-jet so that a battered end is not needed. A steel pipe or bar, of about 1½ in. outside diameter, is first driven and the first pile is placed with a jet so that one groove fits against the bar. In the other groove a second bar is placed, and this is followed by further piles with bars between them. The bars are pulled out as soon as possible, and are replaced immediately by a dry creosoted wooden stick of slightly smaller diameter. When the stick becomes wet it fills the space completely and so prevents leakage of fine sand. The upper part of the stick decays in ten years, but by then the cores are filled with roots of grass and moss. The lengths of the piles are limited by the lengths of the sticks and generally only two sizes are made, namely, 3 in. by 1 ft. 8 in. by 9 ft. 10 in. and 4 in. by 1 ft. 8 in. by 13 ft. 1½ in.

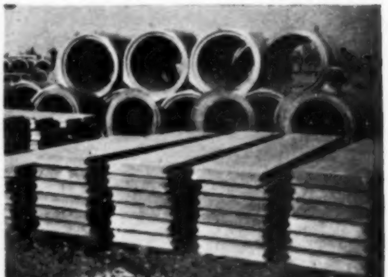


Fig. 7.

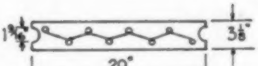


Fig. 8.

Due to the mass production of these types, special reinforcement cages have

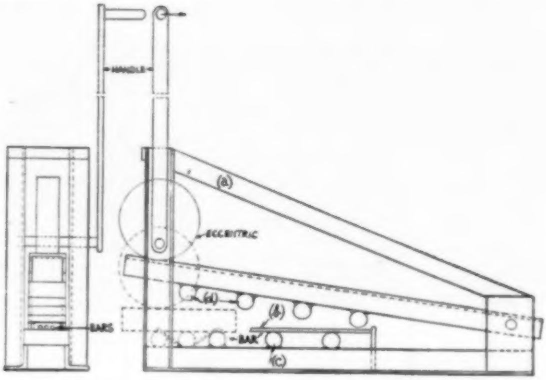


Fig. 9.—Reinforcement Crimping Machine.

been developed which give economy in steel and labour. The ¼-in. links are crimped (*Fig.* 8) several at a time in a special press. The machine used for crimping the wires is shown diagrammatically in *Fig.* 9. The handle turns an eccentric wheel (which is greased) which forces the lever (*a*) to the lower position. The bars to be bent (*b*) are placed on the rolls (*c*), which are round steel bars or pipes. Similar rolls (*d*) on the lever are staggered between the rolls (*c*) so that when the lever is pushed down the wires are crimped as shown by dotted lines. A helical spring is used to keep the handle in the raised position while the crimped bars are removed and more straight bars are placed in the machine.

The links are electrically welded to the longitudinal bars, and circular bar-spacers are added. The bars are welded to the apexes of the bends in the links. Reinforcement for the shorter piles comprises eight $\frac{5}{16}$-in. bars and thirteen ¼-in. links. For the longer piles the reinforcement is eight ⅜-in. bars and seventeen ¼-in. links. The piles are slightly tapered, eliminating the need for special wedge-shape piles. At a distance of 6 in. from both ends 1½-in. holes are formed to facilitate lifting and placing; these holes are later filled with mortar.

Fig. 10 shows a small bridge with abutments formed of tongue-and-groove sheet piling with special corner posts. This type of bridge can be built without interruption to the flow of the water and is often used in wet areas.

Fig. 10.

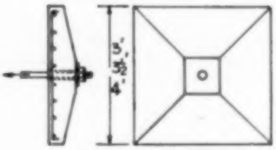


Fig. 11.

Most specifications require the concrete for sheet piles to have a compressive strength of 5000 lb. or 6500 lb. per square inch on test cubes. Much higher compressive strengths are obtained in many cases, sometimes up to 12,000 lb. or 13,000 lb. per square inch.

Anchors for Sheet Piles.

Anchors for sheet-piled walls are generally of the shape shown in *Fig.* 11. The slabs have one, or sometimes two, meshes of ⅜-in. bars at 4 in. to 6 in. centres. The

anchor-bars are $1\frac{1}{2}$ in. or 2 in. diameter and are connected to cast-in-situ beams or copings on top of the sheet piles. The steel is covered with bituminous paint or other protection. The tops of sheet piles which are to be driven with a hammer have a small projection with a curved top to form a shoulder to receive a well-fitting rope and wooden cushion.

Extensions for Wooden Piles.

Figs. 12 to 15 show a type of concrete pile, known as "oplangers," which are used to extend and to continue the driving of wooden piles. These are of various patented types. Their advantages are: (*a*) Long piles to carry light loads are cheaper in a wood-concrete combination

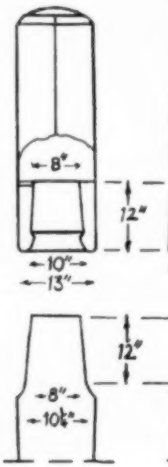


Fig. 12.

Fig. 13.

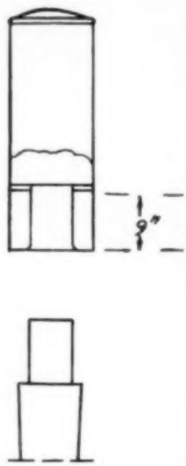


Fig. 14.

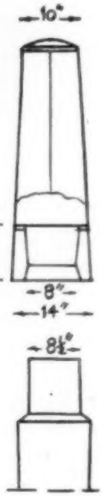


Fig. 15.

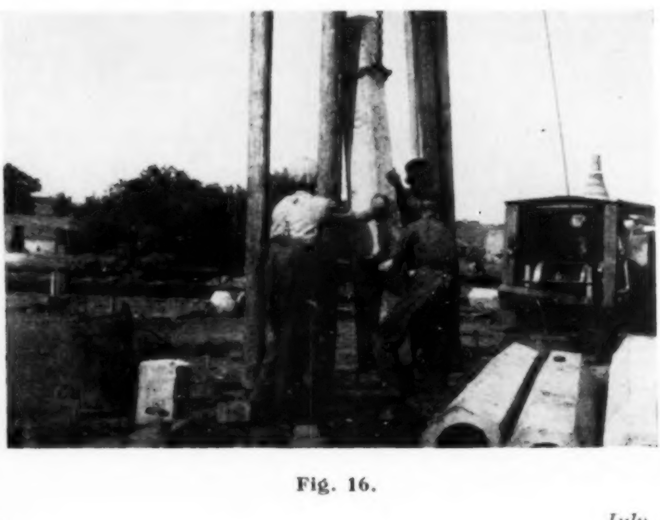


Fig. 16.

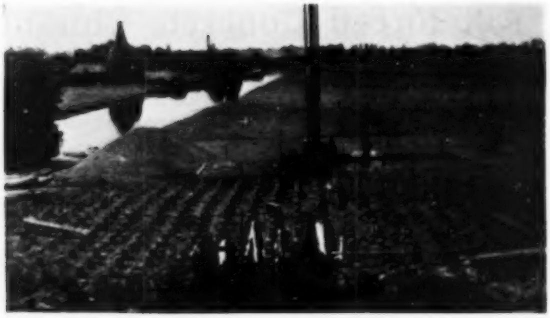


Fig. 17.

than in all-concrete construction; (*b*) the pile-driver can be lighter and shorter; (*c*) wood is less affected than Portland cement concrete by chemicals in the soil. The shapes of the bottoms of some types of these extensions are shown in *Figs.* 12 to 15, together with the shapes of the tops of the wooden piles. The shapes shown in *Figs.* 12 and 15 are designed to lock the concrete and wood together. The concrete extension in *Fig.* 12 is driven so that it locks on the tapered head of the wooden pile, while in *Fig.* 15 the head of the pile is cylindrical so that the wood is slightly compressed when the extension is driven on it. When the ground water reaches the joint, the wood expands and fills the dovetail shape in the extension shown in Fig. 15. *Figs.* 13 and 14 are similar designs; the wood is compressed in *Fig.* 13 resulting in a slightly stronger joint, although this may stop the circulation of water through the wood. To prevent the formation of an air-cushion in the joint, vents are provided as shown in the drawings.

These concrete extensions are hexagonal, octagonal, or circular in cross section. They are generally made in lengths of from 3 ft. to 12 ft., but sometimes up to 18 ft. Other common dimensions are given in the illustrations. The concrete extensions cannot be used for sloping piles. *Fig.* 16 shows a concrete extension being placed on a wooden pile, and *Fig.* 17 shows a foundation in which these composite piles are used.

The reinforcement is often welded and consists of eight or more bars of $\frac{3}{8}$ in. or $\frac{1}{2}$ in. diameter. The helical binding is in most cases $\frac{1}{4}$ in. in diameter and is very closely-spaced around the joint at the bottom, where there are often two helices of $\frac{3}{8}$-in. or $\frac{1}{2}$-in. bars.

Lecture Courses on Roads.

LECTURE courses will be held at the Road Research Laboratory, Harmondsworth, during the autumn and winter of 1952–53 on the properties of road materials and their application. The courses on concrete roads will be held on November 18 to November 27 and on December 2 to December 11. A fee of £10 10*s.* will be charged for each course. Forms of application can be obtained from the Director, Road Research Laboratory, Harmondsworth, Middlesex.

A two-days' course on the stabilisation of soil with cement has been arranged, in co-operation with the Cement and Concrete Association, on August 20 and 21 at the Association's Training Centre, Wexham Place, Stoke Poges, Bucks. (near Slough), from which particulars can be obtained (there will be no fee for this course).

Reinforced Concrete Louvres.

ONE of the walls of the new chemistry building at the University of Hongkong will consist entirely of movable reinforced concrete louvres, which can be raised and lowered (like a Venetian blind) when necessary to shield the rays of the sun.

Four Reinforced Concrete Chimneys at Fawley, Hants.

THE initial scheme for the new Esso refinery at Fawley, on Southampton Water, allowed for three reinforced concrete chimneys 250 ft. high above ground, and it was later decided to include a fourth chimney of the same height. One chimney serves the boiler plant, another the atmospheric heater, another the atmospheric and vacuum heaters, and the fourth (*Fig.* 1) also serves the boiler plant. The chimneys were constructed by Custodis (1922), Ltd., of London, the first three being designed by the Custodis Construction Company of New York and the fourth by Custodis (1922), Ltd., London, in each case in co-operation with Foster Wheeler, Ltd., who were the designers and engineers for the refinery.

The chimneys were designed in accordance with the American standards, which have been the basis of design for most of the very large chimneys in America for the past fifteen years, modified by the provisions of the British Standard Code of Practice, under which the design is based on a wind velocity of 78 miles per hour, and the building by-laws of the Rural District Council which require a wind load of 25-lb. per square foot to be assumed on the vertical projection for the full height. The possibility of earthquake was not considered. The main design data were: Weight of concrete, 150 lb. per cubic foot; weight of firebrick in place, 130 lb. per cubic foot; compressive strength of concrete at 28 days, 3750 lb. per square inch; atmospheric temperature, 20 deg. F.; maximum temperatures—boiler chimneys, 650 deg. F.; atmospheric heater chimney, 900 deg. F.; atmospheric and vacuum heater chimney, 1000 deg. F.

The chimneys taper from bottom to top. The thickness of the walls and the vertical reinforcement were designed to resist the dead load, the wind, and temperature effects, and the circumferential reinforcement was designed to resist diagonal tension and horizontal temperature effects.

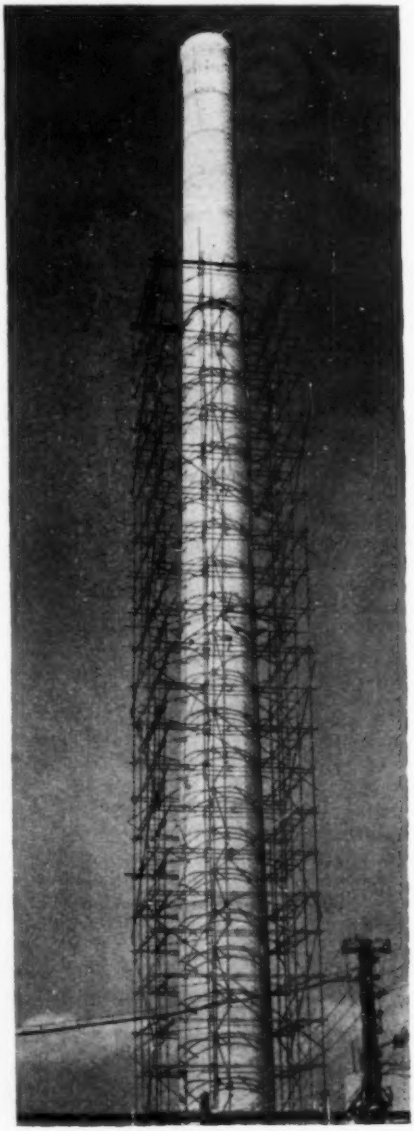


Fig. 1.

The Shafts.

The diameter inside the lining is 8 ft. at the top and 11 ft. at the bottom in the

case of the first boiler chimney, and 9 ft. and 12 ft. respectively in the case of the other three chimneys. In all the chimneys the lining is 4½ in. thick at the top and 18 in. thick at the bottom; there is a minimum air-space of 4¼ in. between the shaft and the lining. In all cases the concrete is 6 in. thick at the top, increasing to 12 in., 13 in., or 14 in. at the bottom.

The reinforcement of the first boiler chimney comprises 32 vertical ½-in. bars at the top increasing to 124 ⅞-in. bars at the bottom, and ½-in. bars at 12-in. centres circumferentially. The second boiler chimney has 34 vertical ⅝-in. bars at the top increasing to 140 ¾-in. bars at the bottom and circumferential reinforcement of ⅝-in. bars at 12-in. centres. The other two chimneys have 34 vertical ½-in. bars at the top increasing to 164 ¾-in. bars at the bottom and ½-in. circumferential reinforcement at 12-in. centres in the case of the chimney for the atmospheric heater and at 10-in. centres in the chimney for the atmospheric and vacuum heaters. There is additional vertical, diagonal, and circumferential reinforcement at flue openings, of which there are two in each of the boiler chimneys and vacuum-heater chimney, and one in the atmospheric-heater chimney. The total amounts of reinforcement and concrete in the shafts are respectively 15·1 tons and 259 cu. yd. in the first boiler chimney, 16·5 tons and 284 cu. yd. in the atmospheric-heater chimney, 17·6 tons and 282 cu. yd. in the vacuum-heater chimney, and 18·75 tons and 307 cu. yd. in the second boiler chimney.

Foundations.

The foundations of all the chimneys are designed for an earth pressure of 2 tons per square foot at 7 ft. below ground level, where bores indicated a stratum of fine silica sand. In the case of the first boiler chimney pockets of clay were encountered and another 10 ft. were excavated, the additional excavation being filled with mass concrete on which the foundation was constructed. All the foundations are octagonal in plan and reinforced top and bottom with vertical bars tied to the reinforcement in the shafts.

The total weight on each foundation is about 1000 tons, and the sizes of the foundations are 38 ft. wide by 8 ft. thick (excluding the sub-base of the shaft) for the first boiler chimney and 39 ft. 6 in. wide and 7 ft. 6 in. thick for each of the other three chimneys. The amount of reinforcement in the foundations is about 17½ tons for the first boiler chimney, 12½ tons for the second boiler chimney, and 10½ tons for each of the other two chimneys.

Lining.

A lining of Glenboig (for three) and Craigend (for one) fire-bricks set in fire-clay was used, except for the top 35 ft. which was constructed entirely of American acid-proof bricks laid in acid-resisting putty. The upper 6 ft. 6 in. of the shaft of each chimney was constructed entirely of acid-resisting bricks 9 in. thick, because the acid resulting from the combination of the oxides of sulphur with atmospheric or condensed moisture under adverse weather or temperature conditions is liable to cause damage to concrete and firebrick. The linings are reinforced with 3-in. by ¼-in. bands spaced at different intervals in the case of the chimneys serving the boilers and those serving the atmospheric heater and atmospheric and vacuum heaters, and in the former case the bands are covered with sheet lead. In the case of the chimneys for the atmospheric heater and the vacuum heater the bands in the upper 35 ft. are of stainless steel. The steel channels which bridge the bands at the flue openings in the boiler chimneys are covered with lead; in the other chimneys these channels are of light steel alloy. The openings to permit circulation of air in the annular space between the lining and the shaft are at the bottom of the outer shafts and at a height of 150 ft. in each chimney, the top of the annular space being left open to the atmosphere.

Fittings.

At the top of each chimney cast-iron caps ¾ in. thick cover the lining and the outer shaft. The sections of the caps are bolted together with stainless-steel bolts. The cap is designed to permit differential expansion of the lining and the shaft.

Access to the top of each chimney is by ⅞ in. diameter galvanised step-irons 15 in. wide spaced at 12 in. centres vertically. Access to the inside of the chimneys is through 24-in. by 36-in. cast-iron clean-out doors. A curtain wall is provided in the linings to protect the doors from gases.

The lightning conductors comprise six $\frac{5}{8}$-in. diameter monel-metal vertical projection rods and two 1-in. by $\frac{3}{16}$-in. copper-tape conductors on opposite sides of each chimney, the upper 25 ft. of the tapes being covered with $\frac{1}{16}$-in. lead.

Four lights are installed at 150 ft. above ground and four at 240 ft. above ground as warnings to aircraft; access to the lights is by step-irons.

Construction.

The shaft of the first chimney was commenced on May 24, the second on July 19, and the third on September 7, 1950. The shafts were completed on August 25, October 24, and December 2, 1950, respectively. Immediately following completion of the shafts the brick tops were constructed, and the firebrick lining of the first chimney was commenced on March 25, the second on April 29, and the third on May 5, 1951. The first chimney was completed on April 29, the second on June 14, and the third on June 28, 1951, about four months ahead of the schedule. The last chimney was commenced in July, 1951; the shell was completed early in October, and the lining was completed by the middle of December.

Prestressed Concrete Sheet Piles.

Prestressed concrete sheet piles were used at the new iron ore quay at Tyne Dock to form a curtain wall. The piles are 40 ft. long and $23\frac{7}{8}$ in. by 9 in. in cross section. The tongues and grooves on the sides are designed to permit the introduction of colloidal grout between the piles after driving. There are 84 0·2-in. diameter high-tensile steel wires in each pile; in addition, mild steel is provided in the head and toe and stirrups are provided throughout the length of the pile. The head is slightly reduced in cross section, and the toe is pointed and bevelled so that each pile is forced against its neighbour during driving. No shoes are fitted. The piles were manufactured at the Tallington works of Messrs. Dow-Mac (Products), Ltd. Each pile (*Fig.* 1) weighs approximately 4 tons, and the lifting points were marked before despatch.

Fig. 1.—Lifting a Pile.

The driving was done by the main contractors, the Yorkshire Hennebique Contracting Co., Ltd., with a 3-tons single-acting hammer with a drop of 4 ft. Bolt-holes in the piles were not permitted, and the piles were held in the leaders by means of clamps. Hard driving conditions were experienced, and up to 150 blows per foot of penetration were required in many cases; the damage to the heads of the piles was, however, negligible. The piles were designed by Mr. R. B. Porter, M.I.C.E., chief engineer of the Tyne Improvement Commission; the consulting engineers were Messrs. Rendel, Palmer & Tritton.

Pressure-Type Air Meters for Air-Entrained Concrete.*

BY P. J. F. WRIGHT, B.Sc., and A. D. McCUBBIN

OF THE ROAD RESEARCH LABORATORY, DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH.

THIS article describes tests carried out at the Road Research Laboratory on three pressure-type "air meters" for measuring the air content of air-entrained concrete. In these meters a known pressure is applied to a given volume of freshly-mixed concrete and the resultant decrease in volume of the concrete is measured. The method has been fully described elsewhere [1] [2] and is illustrated diagrammatically in *Fig.* 1. The air content indicated by each meter was compared with the air content obtained by a gravimetric method.[2] [3]

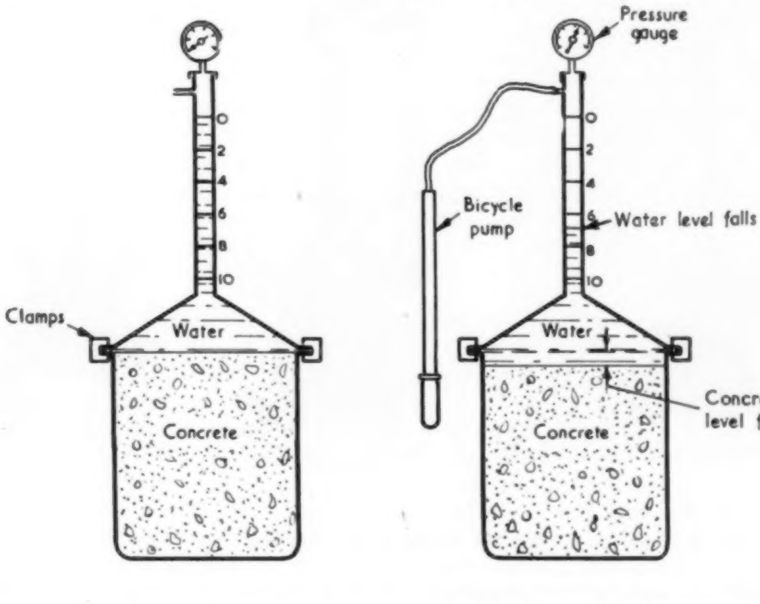


Fig. 1.—Diagram of Pressure-type Apparatus for Determining the Air Content of Air-entrained Concrete.

The principal purpose of entraining air in concrete is to improve its resistance to frost. The workability is also improved, but this is accompanied by a reduction in compressive strength which, if the water-cement ratio is constant, amounts to about 5½ per cent. for each 1 per cent. by volume of air. If, however, the water-cement ratio is reduced so that the workability remains constant, and other minor modifications are made to the proportions of the concrete, air may

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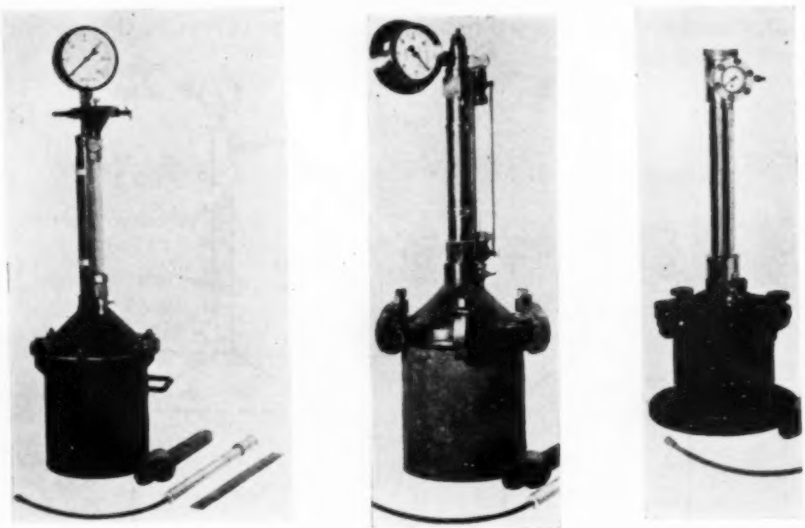
be entrained with little or no loss of strength. The amount of air entrained varies with different cements, proportions, and aggregates, and with the temperature, and it must be controlled within limits (frequently 4 per cent. to 6 per cent. by volume); if the air content is too high the strength will be seriously reduced, while if it is too low the workability will be reduced. Thus a simple and reliable instrument is desirable for measuring the air content at regular intervals, and the pressure-type meter has found favour in America. An instrument of this type has recently been made at the Road Research Laboratory,



No. 1. No. 2. No. 3.

Fig. 2.—Types of Air Meters.

and before it was put into general use its performance was compared with that of other available pressure-type air meters and with the gravimetric method previously used.

Details of the Air Meters.

The air meters used were as follows. (1) A proprietary air meter, with a capacity of about $\frac{1}{4}$ cu. ft. of concrete, designed to measure from 0 to 7·5 per cent. of air. This instrument was received already calibrated in percentage air content when operating at a pressure of 15 lb. per square inch. (2) An air meter made at the Building Research Station, with a capacity of about $\frac{1}{4}$ cu. ft., designed to measure up to 8 per cent. of air. (3) An air meter made at the Road Research Laboratory, with a capacity of about $\frac{1}{10}$ cu. ft., designed to measure up to 9 per cent. of air. Meters (2) and (3) were calibrated at the Laboratory at an operating pressure of 15 lb. per square inch. The more common method is to use a pressure such that the graduated scale indicates the air-content directly, but for these

tests a pressure of 15 lb. per square inch was used for all the meters. The three air meters are illustrated in *Fig.* 2.

Scope of Tests.

One mixture was used in the proportions of 1 : 7½ by weight of dry materials, using normal Portland cement, graded river gravel aggregate of ¾ in. maximum size, and river sand, the sand comprising one-third of the total aggregate. In order to produce a mixture suitable for hand compaction a water-cement ratio of 0·67 was used; from preliminary tests this was found to give a compacting factor of 0·95 when about 5 per cent. of air was entrained.

Vinsol-resin was used to entrain the air, a 10 per cent. pre-neutralized solution in sodium hydroxide being added to the mixing water just before mixing. The quantity of solution was such that the weight of Vinsol-resin was 0·015 per cent. of the weight of cement, and this entrained an average of about 6 per cent. of air. The concrete was mixed in an open-pan paddle-type mixer of 1 cu. ft. capacity; twelve batches were made and on each batch four determinations of the air content were made, three by the air meters and one by the gravimetric method. Two operators carried out the tests and each made two of the determinations on each batch. The operators changed instruments after each batch. Two additional batches were made in the same proportions but with the quantity of Vinsol-resin reduced to give about 2 per cent. of entrained air; tests were made on these mixtures with two of the air meters and by the gravimetric method.

Methods of Determining the Air Content.

Gravimetric Method.—The vessel used was the cylinder of a compacting-factor apparatus, which has a volume of about ⅛ cu. ft. The concrete was compacted by hand in 2-in. layers with 25 blows of a standard 4-lb. punner. The cylinder was filled level and screeded off. It was then weighed to the nearest 5 gm. and the air content was found by comparing the weight of air-entrained concrete with the theoretical weight of the same volume of concrete free from air, calculated from the known proportions and densities of the constituents.

Pressure Method.—For each instrument the concrete was compacted as for the gravimetric method, the number of blows of the punner being proportional to the volume, assuming that 150 blows were appropriate for a volume of ⅛ cu. ft. When the vessel was full and screeded off, a thin circular metal plate was placed on top of the concrete to prevent the water eroding the surface of the concrete, the top section was clamped down tight and filled with water to the top mark on the scale. Air pressure was applied with a bicycle pump up to 16 lb. per square inch; the pressure was then found to drop slowly (probably due to the porosity of the aggregate), and when the pressure gauge recorded 15 lb. per square inch the water level was read to the nearest 0·1 per cent. of air content. The air pressure was then released. The water level did not fully return to zero owing to the effect of the porosity of the aggregate; the small correction was noted and subtracted from the gross air content.

Results of Tests.

The results of the tests are given in *Table* I. The average air contents of the batches varied considerably; larger variations might be expected on a con-

structional site if no effort were made to control the air content. The agreement between different instruments was good, and a statistical analysis of the results showed that there was no general difference between the values obtained with different instruments, although No. 4 gave higher values than No. 1. The difference here may have been due to a calibration error in No. 1 or to a real difference between the gravimetric and pressure methods. This suggests that it would be

TABLE I.—RESULTS OF MEASUREMENTS OF ENTRAINED AIR.
(Figures are percentages of air by volume.)

Meter		1 (Proprietary)	2 (B.R.S.)	3 (R.R.L.)	4 (Grav.)	Average
Batch	1	3·5	3·6	3·6	3·5	3·55
	2	5·8	5·4	5·8	5·8	5·7
	3	6·1	6·7	6·7	7·1	6·65
	4	6·6	7·2	6·4	7·8	7·0
	5	5·6	5·3	5·7	5·7	5·6
	6	5·6	5·8	5·5	5·7	5·65
	7	6·7	6·2	6·5	6·7	6·5
	8	6·5	6·4	6·7	6·7	6·6
	9	5·3	6·1	5·5	6·2	5·8
	10	5·3	5·3	5·6	5·6	5·45
	11	5·2	5·9	5·8	6·1	5·75
	12	5·4	6·3	6·4	5·9	6·0
Average		5·6	5·85	5·85	6·1	5·85
Standard deviation within batches .		0·28	0·23	0·25	0·25	
	13	1·5	—	1·7	1·7	1·63
	14	1·4	—	1·5	1·8	1·57

advisable to check the calibration of any instrument before use. The analysis did not show any difference in the uniformity of results obtained with different instruments.

The time taken to carry out the tests was about five minutes for each determination. The gravimetric determination was the quickest since the time required to weigh the vessel was less than that required to complete the pressure operation; however, the calculation of the gravimetric result takes longer, additional apparatus (a balance) is required, and accurate information regarding proportions and specific gravity is needed. Meter No. 3 was the quickest to use of the pressure meters, owing to the relatively small volume of the concrete and to the ease of filling the instrument with water. The test with meter No. 1 took longest, mainly owing to the presence of a plate perforated with small holes in the filling funnel which increased the time required to fill the meter with water. The plate was intended to prevent stones entering the meter and becoming lodged in the gauge-tube, but this might have been achieved more satisfactorily by using gauze and a larger hole or holes. The hinged clamping screws of meters Nos. 1 and 3 were preferred to the loose G-clamps of meter No. 2 for fastening down the top section. Meter No. 2 had been designed with G-clamps so that the

cylinder could be used more easily for determinations of density and similar purposes.

Conclusions.

It was found that, under the test conditions, the pressure and gravimetric methods both gave reliable results. For site conditions the pressure method is preferred, as it does not require an accurate balance, is simple to use, and provides a direct reading ; also it does not require an accurate knowledge of the proportions of the concrete, which is normally lacking on the site owing to inevitable variations. The $\frac{1}{10}$ cu. ft. cylinder of meter No. 3 was satisfactory with $\frac{3}{4}$-in. maximum size aggregate.

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(1) Menzel, C. A. Procedure for determining the air content of freshly mixed concrete by the rolling and pressure methods. *Proc. Am. S.T.M.*, 1947, 47, 833.

(2) Klein, A., Pirtz, D., and Schweizer, C. B. Comparison of three methods of measuring air-entrainment in concrete. *Proc. Am. S.T.M.*, 1947, 47, 876.

(3) American Society for Testing Materials. Standard method for test for weight per cubic foot and air content (gravimetric) of concrete. Am. S.T.M. Designation C138–44. A.S.T.M. Standards, 1944. Part II. Non-metallic materials—constructional, pp. 487–9. Philadelphia, Pa. 1944 (Am. S.T.M.).

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